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Vol. XXIV. MARCH, 1898. No. 3.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publication.

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SOCIETY AFFAIRS.

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ILLUSTRATIONS.

The prices of publications are as follows: Proceedings, $6 per annum; Transactions, $10 per annum. Postage will be added when they are sent to foreign countries.

American Society of Civil Engineers.

OFFICERS FOR 1898.

President, ALPHONSE FTELEY.

Vice-Presidents.

Term expires January, 1899:	*Term expires January, 1900:*
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JOHN F. WALLACE.	FREDERIC P. STEARNS.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOHN THOMSON.

Directors.

Term expires January, 1899:	*Term expires January,* 1900:	*Term expires January,* 1901:
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WM. BARCLAY PARSONS,	RUDOLPH HERING,	JOHN KENNEDY,
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THOMAS W. SYMONS.	LORENZO M. JOHNSON.	GEORGE Y. WISNER.

Assistant Secretary, T. J. McMINN.

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THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

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JOHN R. FREEMAN,	JOHN F. WALLACE,	FREDERIC P. STEARNS,
DANIEL BONTECOU.	HENRY S. HAINES.	JOHN KENNEDY.

Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open from 9 to 22 o'clock every day, except on Sundays, when the hours are from 14 to 19 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - 2236-38th Street.
CABLE ADDRESS, - - - - "Ceas, New York."

Vol. XXIV. MARCH, 1898. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

Wednesday, March 2d, 1898.—The meeting was called to order at 20.20 o'clock, President Alphonse Fteley in the chair; Charles Warren Hunt, Secretary, and present, also, 75 members and 11 visitors.

The minutes of the meetings of February 2d and 16th, 1898, were approved as printed in *Proceedings* for February, 1898.

A paper by Joseph Mayer, M. Am. Soc. C. E., entitled "The Economic Depth for Canals of Large Traffic," was presented by the author. Correspondence on the subject from Messrs. J. P. Frizell, G. H. Raymond and Thomas W. Symons was presented by the Secretary. The paper was discussed orally by Messrs. W. P. Craighill, E. Sherman Gould, George S. Morison, Edward P. North and Emerson Foote.

Ballots were canvassed, and the following candidates declared elected :

As Members.

John Brunner, Pittsburgh, Pa.
Richard Sutton Buck, Niagara Falls, N. Y.
Otis Francis Clapp, Providence, R. I.
Edmund Burns Geddes, Natchez, Miss.
Charles Lewis Harrison, Lockport, N. Y.
Horace Joseph Howe, Medford, Mass.
Archibald Olin Powell, St. Paul, Minn.
Ernest Kay Scott, Chicago, Ill.
Karl Spörck, Kansas City, Mo.
Arthur Smith Tuttle, Brooklyn, N. Y.
Aaron Twyman, Pullman, Ill.

As Associate Members.

Henry Wilde Hayes, Fitchburg, Mass.
Percy Holbrook, New York City.
John T. Noye Hoyt, New York City.
Jere Chamberlain Hutchins, Detroit, Mich.
George Evarts Low, Brooklyn, N. Y.
Richard McCulloch, St. Louis, Mo.
Thomas McElderry Vickers, Syracuse, N. Y.

The Secretary announced the election by the Board of Direction on March 1st, 1898, of the following candidate :

As Junior.

Edgar Tweedy Belden, Stamford, Conn.

The Secretary announced the death of Randall Hunt, elected Member May 2d, 1883 ; died January 24th, 1898.

March 16th, 1898.—The meeting was called to order at 20.20 o'clock, Vice-President Edward P. North in the chair ; Charles Warren Hunt, Secretary, and present, also, 88 members and 18 guests.

A paper by B. F. Thomas, M. Am. Soc. C. E., entitled "Movable Dams," was presented by the Secretary, together with correspondence on the subject from Messrs. J. P. Frizell, H. M. Chittenden, Arthur M. Bowman and D. A. Watt. The paper was discussed orally by Messrs. R. B. Stanton, F. S. Washburn and L. L. Buck.

The Secretary announced the death of William Starke Rosecrans, elected Member July 5th, 1882; died March 11th, 1898.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

March 1st, 1898.—President Fteley in the chair, Chas. Warren Hunt, Secretary, and present, also, Messrs. Deyo, Hering, Just, Morison, North, Owen, Parsons, Schneider, See and Thomson.

The Secretary was directed to forward a mailing list of the Society to the Chief of Engineers, U. S. A., with a request that a copy of Major Symons' report on the canal question be forwarded to each member of the Society.

A letter was presented from the President and Secretary of The Institution of Civil Engineers, with regard to proposed courtesies to be extended by that Institution to members of this Society who may visit England during the International Exhibition, to be held in Paris in 1900. A suitable reply was arranged for.*

The Library Committee was requested to secure a set of photographs of the exterior and interior of the New Society House.

Action was taken in regard to a division of the territory occupied by the membership of the Society into seven geographical districts, as required by Art. VII, Sec. 1, of the Constitution.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society will be open every day hereafter from 9 to 22 o'clock, except on Sundays, when the hours will be from 14 to 19 o'clock.

INTERNATIONAL EXHIBITION, PARIS, 1900.

For the benefit of members who contemplate visiting the International Exhibition, to be held in Paris in 1900, the following correspondence is published.

* See page 60.

THE INSTITUTION OF CIVIL ENGINEERS,
Great George Street, Westminster, S. W.,
1 FEBRUARY, 1898.

TO BENJAMIN MORGAN HARROD, ESQ., PRESIDENT, AND THE COUNCIL OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, 127 EAST TWENTY-THIRD STREET, NEW YORK, U. S. A.

Gentlemen,—In view of the intended International Exhibition in Paris, in the year 1900, it has appeared to the Council of this Institution not unlikely that some of the members of your Society may visit Europe in a more or less organized party in that year.

We are desired by the Council of this Institution to say that, should such a step be taken, and should your members be able to visit England, the Institution of Civil Engineers would wish to welcome its professional brethren of the United States with a warm greeting, to receive them at the house of the Institution, and in such other ways as may be found agreeable to take advantage of such a valued opportunity of testifying their regard for the members of your Society.

We have the honor to be, gentlemen,

Yours very faithfully,
J. WOLFE BARRY,
President.
J. H. T. TUDSBERY,
Secretary.

AMERICAN SOCIETY OF CIVIL ENGINEERS,
220 West 57th Street,

NEW YORK, MARCH 3d, 1898.

TO SIR J. WOLFE BARRY, PRESIDENT, AND J. H. T. TUDSBERY, D. SC. SECRETARY OF THE INSTITUTION OF CIVIL ENGINEERS, GREAT GEORGE STREET, WESTMINSTER, S. W., LONDON.

Gentlemen,—Your communication of February 1st, 1898, addressed to the President and Council of the American Society of Civil Engineers, extending the courtesies of the Institution of Civil Engineers to our members on the occasion of the intended International Exposition in Paris, in the year 1900, was acknowledged by the Secretary upon its receipt, and was presented on the 1st inst. to the Board of Direction of the Society.

It is impossible to state at this early date whether a party will be organized for a visit to the Exposition, but without doubt many of our members will visit Europe at that time, and our membership will be duly notified of your kind invitation.

On behalf of the Board of Direction, which directed us so to do, and of our members, we beg to return our sincere thanks for your courteous invitation, and to assure you that those of our members who are fortunate enough to be able to visit England in 1900, whether in an organized party or in their individual capacity, will be most happy to avail themselves of your hospitable offer.

Yours very sincerely,
A. FTELEY,
President.
CHAS. WARREN HUNT,
Secretary.

ENGINEERING COMPETITION.

The Secretary has received from Mr. August Peterson, *Vice-Consul, Sweden and Norway*, a notification that competitive designs are invited for the arrangement of new Railroad Stations, Junctions, etc., for the City of Stockholm.

The first prize is to be 12 000 Swedish crowns (about $3 230); the second, 8 000 Swedish crowns (about $2 150) and the third, 4 000 Swedish crowns (about $1 075).

The time for competition will expire at noon on August 31st, 1898.

Particulars concerning the nature of the work will be furnished by the Swedish-Norwegian Legation, 2011 Q Street, Washington, D. C., or by the Vice-Consul of Sweden and Norway, Mr. August Peterson, Le Droit Building, corner of F and Eighth Streets, Washington, D. C.

Security to the amount of $13.50 for the use of drawings is required, but this will be refunded when the drawings are returned.

MEETINGS.

Wednesday, April 6th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by George Hill, Assoc. M. Am. Soc. C. E., entitled "Steel Concrete Construction," will be presented. It is printed in this number of *Proceedings.*

Wednesday, April 20th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by James Ritchie, M. Am. Soc. C. E., entitled "The Construction of the Lorain Dry Dock and Shipyard of the Cleveland Ship-Building Company," will be presented. It is printed in this number of *Proceedings.*

Wednesday, May 4th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by H. N. Ogden, Jun. Am. Soc. C. E., entitled "Flushing In Pipe Sewers," will be presented. It is printed in this number of *Proceedings.*

DISCUSSIONS.

Discussion on the paper by Joseph Mayer, M. Am. Soc. C. E., entitled "The Economic Depth for Canals of Large Traffic," which was presented at the meeting of March 2d, 1898, will be closed April 15th, 1898.

Discussion on the paper by B. F. Thomas, M. Am. Soc. C. E., entitled "Movable Dams," which was presented at the meeting of March 16th, 1898, will be closed May 1st, 1898.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.

			Date of Membership.
John Brunner	278 Shady Ave., Pittsburg, Pa.		Mar. 2, 1898
Otis Francis Clapp	City Engineer, Providence, R. I.		Mar. 2, 1898
Edmund Burns Geddes	U. S. Asst. Eng., Natchez, Miss.		Mar. 2, 1898
Charles Lewis Harrison	P. O. Box 1688, Rome, N. Y.		Mar. 2, 1898
Ernest Kay Scott	901 Security Bldg., Chicago, Ill.		Mar. 2, 1898
Arthur Smith Tuttle	Asst. Eng., Dept. of Water Supply, Room 41, M'cipal Bldg., Brooklyn, N.Y.	Jun.	Mar. 2, 1887
		Assoc. M.	May 2, 1894
		M.	Mar. 2, 1898

ASSOCIATE MEMBERS.

Henry Wilde Hayes	25 Prospect St., Fitchburg, Mass.		Mar. 2, 1898
Jere Chamberlain Hutchins	12 Woodward Ave., Detroit, Mich.		Mar. 2, 1898
George Evarts Low	12 Downing St., Brooklyn, N.Y.	Jun.	Nov. 6, 1894
		Assoc. M.	Mar. 2, 1898
Richard McCulloch	3800 Easton Ave., St. Louis, Mo		Mar. 2, 1898
George MacLeod	29th and High Sts., Louisville, Ky.		Sept. 1, 1897
James Clark McGuire	26 Cortlandt St., New York City		Jan. 5, 1898
Thomas McElderry Vickers	113 N. State St., Syracuse, N. Y.		Mar. 2, 1898

JUNIORS.

George Heckman Burgess	Care of Penna. Co., Pittsburg, Pa.	Feb. 1, 1898
Edward Joseph Carney	53 West 68th St., New York City	Feb. 1, 1898
Charles Derleth, Jr.	674 East 135th St., New York City	Feb. 1, 1898
Leon Elie Lion	1010 Burgundy St., New Orleans, La.	Feb. 1, 1898

CHANGES AND CORRECTIONS.

MEMBERS.

WALTER LINSLEY COWLES.........1204 Sixth Ave., Beaver Falls, Pa.
CASPAR WISTAR HAINES...........Care of Silva & Haines, Engrs., Porfirio Diaz 10½, Puebla, Mexico.
WILLIAM DUNBAR JENKINS........Chf. Eng. Aransas Pass Harbor Co., Aransas Pass, Tex.
ALEXANDER EDWARD KASTL.......98 Walnut St., Clinton, Mass.
HOWARD GEORGE KELLEY.........Chf. Eng., Minneapolis & St. Louis Ry. Co., Minneapolis, Minn.
ALONZO TYLER MOSMAN...........Asst. U. S. Coast & Geodetic Survey, Los Angeles, Cal.
ALFRED PETRY...................Asst. Eng., Commrs. of Water-Works, City Hall, Cincinnati, O.
WILLIAM DOUGLASS PICKETT......Four Bear, Big Horn Co., Wyo.
FRANKLIN COGSWELL PRINDLE.....Civil Eng., U. S. N., Los Angeles, Cal.
EDWIN THACHER ....906 Columbia Bldg,, Louisville, Ky.
BENJAMIN FRANKLIN THOMAS......Box 670, Pittsburg, Pa.
JOHN FINDLEY WALLACE..........Asst. Second Vice-Pres., Ill. Cent. R. R., Central Station, 1 Park Row, Chicago, Ill.

ASSOCIATE MEMBERS.

DUNCAN LEE DESPARD ....1029 Walnut St., Philadelphia, Pa.
RICHARD KHUEN, Jr..............Care of Niagara Falls & Clinton Bridge Co., Niagara Falls, N. Y.
WILLIAM WILLARD LOCKE........ 3 Mt. Vernon St., Boston, Mass.
CHARLES HEDGES MCKINSTRY......Drawer 521, Key West, Fla.
HEW MILLER....................Shore Road and 71st St., S. Brooklyn, N. Y.
OLAF EINAR MOGENSEN...........444 North Clark St., Chicago, Ill.
FRANK HERBERT SNOW............1120 Tremont Bldg., Boston, Mass.
JOHN CLARK SPENCER.............214 Fisk St., Pittsburg, Pa.
HENRY MAYNADIER STEELE........Central Ry. of Georgia, Savannah, Ga.

JUNIORS.

FREDERICK AURYANSEN...........Care Allentown Rolling Mills, Allentown, Pa.
ALFRED CARROLL BELL...........515 Guaranty Loan Bldg., Minneapolis, Minn.
EDWIN JAMES BEUGLER...........18 Elmwood Ave., Bridgeport, Conn.
CHARLES GARTENSTEIG............112 East 81st St., New York City.
VAN ALEN HARRIS.................City Eng., Times Building, Middletown, N.Y.
BENJAMIN FRANKLIN LATTING......152 Wyckoff St., Brooklyn, N. Y.
THOMAS DORSEY PITTS............220 San Pedro Ave., San Antonio, Tex.
CHARLES WINSLOW SHERMAN......Asst. Eng., Sudbury Dept., Met. Water-Works, 3 Mt. Vernon St., Boston, Mass. (Res., 37 Langdon St., Cambridge, Mass.
GRATZ BROWN STRICKLER.........Broadwater, Northampton Co., Va.
ALEXANDER MILLER TODD.........Care of U. S. Engr.'s Office, Greenville, Miss.

ADDITIONS TO

LIBRARY AND MUSEUM.

From Alabama Industrial and Scientific Society:
- Proceedings of the Society for 1897, Vol. VII, Part 2.

From the American Institute of Mining Engineers:
- The Michigan College of Mines.
- The Origin and Mode of Occurrence of the Lake Superior Copper Deposits.
- Some Statistics of Engineering Education.
- The Efficiency of Built-Up Wooden Beams.
- Emery, Chrome-Ore and Other Minerals in the Villayet of Aidin, Asia Minor.
- The Influence of Antimony on the Cold-Shortness of Brass.
- A Study of the Elimination of Impurities from Copper-Mattes in the Reverberatory and the Converter.
- An Automatic Feed-Device for Gas Producers.
- Note on Limonite Pseudomorphs from Dutch Guiana.
- The Ultimate and the Rational Analysis of Clays and Their Relative Advantages.

From L'Association Amicale des Anciens Élèves de l'École Centrale des Arts et Manufactures:
- Annuaire, 1897.

From the Canadian Society of Civil Engineers:
- Address of the President, T. C. Keefer, at the Annual Meeting, January 12th, 1898.

From Commissioners on Topographical Survey, Boston, Mass.:
- Report for the Year 1897.

From Continental Iron Works, New York:
- Catalogue for 1898.

From Alston Ellis, Director:
- Tenth Annual Report of the Agricultural Experiment Station of Colorado for the Year 1897.

From the Engineering Association of the South:
- Proceedings; March, 1898.

From the Engineers' Club of Cincinnati:
- Secretary's Report for the year ending December 23d, 1897.

From R. Feret, Boulogne-Sur-Mer, France:
- Etudes sur la Constitution Intime des Mortiers Hydrauliques.

From P. H. Griffin Machine Works, Buffalo, N. Y.:
- Series of Tests made on January 21st, 1898, on Special Quality Chilled Iron Wheels, made by the New York Car Wheel Works, Buffalo, N. Y.

From Harvard University, Cambridge, Mass.:
- Annual Reports of the President and Treasurer of Harvard College, 1896–97.

From Rudolph Hering, New York:
- Report on the Filtration of the Nuuanu Water Supply of Honolulu, H. I.
- Six copies of "Sewerage Systems."
- Report to Accompanying Plans for the Sewerage and Drainage of Honolulu, H. I.
- Reports of Joint Committee Concerning Park River Sewerage and the General Sewerage System of the City of Hartford.
- Third Annual Report of the Board of Public Works, City of Duluth, Minn., for the year ending February 28th, 1890, including a Report on the Extension of the Sewerage Works, etc., by Rudolph Hering and Andrew Rosewater.
- Report concerning Sewerage System of the City of Hartford, April 17th, 1893.
- Report of the Joint Committee on Water System, etc., of the City of Colorado Springs, 1893.
- Report on a System of Sewerage for the City of Binghamton, N. Y., 1892.
- Annual Report of the Commissioners of Water-Works in the City of Erie.
- Report of the Committee on the Disposal of Garbage and Refuse; reprint from Transactions of the American Public Health Association, 1897.
- Disposal of Sewage.
- Water Purification; reprint from the Journal of the Franklin Institute, February and March, 1895.
- Filtration of Municipal Water Supplies.
- Report on European Sewerage Systems, with Special Reference to the Needs of the City of Philadelphia, 1881.
- Report on a System of Sewerage for the City of New London, Conn., 1885.
- Notes on the Pollution of Streams.
- Report on Sewerage and Out-fall Sewer for Savannah, Ga., 1889.
- Annual Report of the Board of Public Works of the City of Superior, Wis., 1893.
- Report on a Sewerage System for the City of Trenton, N. J., 1885.
- Report on the Disposal of Sewage of the City of Waterbury, Conn., 1896.
- Report on a System of Sewerage for the City of Wilmington, Del., 1883.
- Report on Plans for Sewering and Draining the City of Ithaca, N. Y., 1894.
- Report on Improved Sewerage System, 1884.
- Annual Report of the Board of Street and Water Commissioners of the City of Newark, N. J., 1895.
- Hand-book on the Annexation of Hawaii, by Lorrin A. Thurston.
- The Hawaiian Annual, 1898.
- Hawaii and the Changing Front of the World, by Hon. J. R. Procter.
- Annexation of Hawaii, by Hon. John W. Foster.
- The Hawaiian Islands: Their Resources,

Agricultural, Commercial, and Financial, issued under the Auspices of the Department of Foreign Affairs, 1896.

From Clemens Herschel, New York:
Fourth Biennial Report of the State Engineer to the Governor of Colorado for the years 1887 and 1888, Parts I and II.
Map of the Dominion of Canada.
Ueber den Betrieb auf den Canälen en Nord-America, by Fr. Lange.
Sixth Annual Report of the Department of Docks, New York City, for the Year ending April 30th, 1876.
Annual Message of the President of the Sanitary District of Chicago for the Year ending December 31st, 1894.
Concise Report on the Organization, Resources, Constructive Work, Methods and Progress of the Sanitary District of Chicago.
First Annual Report of the Water Commissioners of the Town of Ware, for the Year ending January 31st, 1887.
Maps of Delaware Bay and River; founded upon a Trigonometrical Survey under the Direction of F. R. Hassler and A. D. Bache.
Sixteenth Annual Report of the Water Commissioners of the City of Taunton, Mass., 1881.
Eighty-fifth Annual Report of the Chief Engineer of the Philadelphia Water Department for the Year 1886.
Report of the Commissioners on Water Supply for the City of Syracuse, N. Y., 1889.
Harbor of New York, Appendix A A of the Annual Report of the Chief of Engineers for 1886.
Appendix A A and Appendix B B of the Annual Report of the Chief of Engineers for 1884.
Appendix C C of the Annual Report of the Chief of Engineers for 1885.
Appendix A of the Annual Report of the Chief of Engineers for 1876.
Appendix G of the Annual Report of the Chief of Engineers for 1891.
Irrigation in the United States, a Report Prepared by Richard J. Hinton, 1887.
Proceedings of the Seventh Annual Convention of the Association of Railway Superintendents of Bridges and Buildings, held in Denver, Colo., October 19th, 20th and 21st, 1897.
Report of the United States Board Appointed to Test Iron, Steel and Other Metals, 1881, Vol. II.
Manual of American Water Works, 1888.
Deeper Waterways from the Great Lakes to the Atlantic: Reports of the Canadian Members of the International Commission, 1897.
Third Biennial Report of the State Engineer of the State of Colorado, for the Years 1885–86.
Report of the State Engineer to the Governor of Colorado, for the Years 1883 and 1884.
Rapport et Mémoire sur le Nouveau Système d'Écluse à Flotteur de M. D. Girard, par Poncelet.
Complete Works of Count Rumford, 4 Vols.
Life of Count Rumford, by George E. Ellis.
Index to the Library of the American Society of Civil Engineers, Part I, 1880.
Annual Report of the Engineer of the City of Newton for the Year 1883.
Report to the Nottingham and Leen Valley Sewerage Board, 1875.
Contributions to our Knowledge of Sewage, by William Ripley Nichols and C. R. Allen.
Complete System of Household and Street Sanitary Engineering Under James Sargent's Patents.
Report of the Committee on Sewerage, transmitting the Report of Phinehas Ball on Disposal of Sewage for the City of Brockton, 1886.
Drainage of Towns and Buildings, by G. Drysdale Dempsey.
Drainage of Districts and Lands, by G. Drysdale Dempsey.

From Frank W. Hodgdon and X. H. Goodnough, Boston, Mass.:
Report of the Joint Board consisting of the Harbor and Land Commissioners and the State Board of Health, upon the Restoration of Green Harbor in the Town of Marshfield, Mass., January, 1898.

From the Institution of Mechanical Engineers:
Proceedings, April, 1897.

From the Iron and Steel Institute, London:
Journal of the Institute for 1897.

From T. C. Keefer, Ottawa, Canada:
Address of the President of the Canadian Society of Civil Engineers, at the Annual Meeting, January 2, 1898.

From Königlichen Technischen Hochschule, Berlin:
Rede zum Geburtsfeste Seiner Majestät des Kaisers und Königs Wilhelm II, in der Aula der Königlichen Technischen Hochschule zu Berlin am 26 Januar, 1898.

From Hew Miller, New York City:
Forty-three Specimens of Different Woods, all from the Interior of British Guiana.

From the New England Cotton Manufacturers' Association:
Transactions of the Semi-Annual Meeting, held October, 1897.

From New England Free Trade League, Boston, Mass.:
Debate on Equitable Protection, Between David Lubin, Esq., and Hon. John E. Russell.

From S. F. Patterson, Secretary:
Proceedings of the Seventh Annual Convention of the Association of Railway Superintendents of Bridges and Buildings, held in Denver, Colo., October 19th, 20th, and 21st, 1897.

From H. V. and H. W. Poor, New York:
Poor's Manual of Railroads for 1897.

From the Roadmasters' Association of America:
Proceedings of the Fifteenth Annual Convention, Held at Old Point Comfort, Virginia, Sept. 14th, 15th and 16th, 1897.

From P. H. & F. M. Roots Co., New York:
Six Copies of Two Separate Tests of Fan and Roots' Blower.

From Rose Polytechnic Institute, Terre Haute, Ind.:
The Rose Technic for January and February, 1898.

From John A. Russell, San Francisco, Cal.:
San Francisco Municipal Reports for the Fiscal Year 1896–97, ending June, 1897.

From Louis Y. Schermerhorn, Philadelphia, Pa.:
Breakwater Construction on the American Coast; Reprinted from the Proceedings of the Engineers' Club of Philadelphia, Vol. xiv, No. 3, October–December, 1897.

From the Smithsonian Institution, Washington, D. C.:
Proceedings and Transactions of the Nova Scotian Institute of Science; Session of 1896–97.

From Traveling Engineers' Association:
Proceedings of the Fifth Annual Convention held in Chicago, Illinois, September 14th, 15th, 16th, 17th, 1897.

From U. S. Treasury Department, Chief of Bureau of Statistics:
Statistical Abstract of the United States, 1897.

From U. S. War Department, Chief of Engineers:
Fifty-two Reports of the Survey of Certain Rivers and Harbors.
Two Copies of a Letter from the Acting Secretary of War, transmitting a Letter from Major-General Miles and Making Recommendations in Regard to the Care and Preservation of the Fortifications and their Armament.
Sixteen Specifications for the Improvement of Certain Rivers and Harbors, the Repairing of Certain Boats, and Electric Lighting Plants.

From the University of Pennsylvania:
Catalogue, 1897–98.

From Henry R. Worthington, N. Y.;
Edison Electric Illuminating Co. of Brooklyn, N. Y.: its Development and its Present and Future Work, By Joseph Wetzler.

BOOK NOTICES.

HIGH MASONRY DAMS.

By E. Sherman Gould, M. Am. Soc. C. E. Boards, 6 x 3¾ ins., pp. 88. New York, D. Van Nostrand Company, 1897. (Van Nostrand Science Series, No. 22.)

This book replaces the original No. 22 of Van Nostrand's Science Series, bearing the same title, by Mr. John B. McMaster.

In the preface the author states that he "is convinced that it is fruitless to attempt to adhere to a general formula for that great desideratum of all economical engineering design, namely, a Section of Equal Resistance. The mathematical researches of those who have investigated this problem have established a vertical section, the basis of which is a right-angle triangle of base equal to two-thirds or three-quarters of its height, as that leading to, or at least looking towards, such a result."

Starting with this fact, the author, by well-known processes, determines the maximum compressive stress upon the material at certain different heights, assuming both an empty and a full reservoir. The dangerous stress in a very high masonry dam being the crushing one, the author has written quite fully upon this particular stress; also on the maximum unit stresses, where the resultant of pressure cuts the base unsymmetrically.

The concluding chapters are on the Construction of High Masonry Dams and Accessories of Dams. There are seventeen figures or diagrams illustrative of the text.

ARITHMETIC OF THE STEAM ENGINE.

By E. Sherman Gould, M. Am. Soc. C. E. Cloth, 7¾ x 5¼ ins., 77 pp. New York, D. Van Nostrand Company, 1897.

The author, in the preface to this work, states that the object of the volume is to furnish a clear and concise digest of the fundamental principles of the steam engine and the practical calculations based upon them. He has not entered into the more abstruse mathematics of the subject but believes that the book contains all that is necessary to solve the ordinary problems relating to steam in its applications to the steam engine. The author's aim has been to put accepted facts in accessible shape for practical use.

The book contains chapters on Heat, Ice and Steam. Under the latter heading are treated: Pressure and Volume, Combustion and Combustibles, the Efficiency of the Steam Engine, Pressures, Horse Power, Indicator Diagrams and Compound Engines. A number of practical examples are given and worked out arithmetically. A table of the Properties of Saturated Steam and a table of Hyperbolic Logarithms conclude the work.

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PAPERS.

CONTENTS:

STEEL CONCRETE CONSTRUCTION.

BY GEORGE HILL, ASSOC. M. AM. SOC. C. E.

TO BE PRESENTED APRIL 6TH, 1898.

It seems hardly necessary to write any prefatory note in connection with a paper on this subject, by reason of the number of papers already submitted, and the decided interest which is universal among engineers in the production of an economical fire-proof material for use under transverse stress. At short intervals there have been submitted for the consideration of the engineer combinations of plastic material and steel or concrete and steel, with the steel in the form of isolated rods and acting entirely in tension; combinations of concrete and steel, in which the steel acts entirely in tension and is distributed throughout the bottom area of the section, and combinations of concrete and steel in which the steel acts under transverse stress; each of which methods is claimed to possess certain advantages.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

The present paper deals with that combination of concrete and steel in which the steel acts entirely in tension, and in which it is distributed through the bottom section of the slab. Theoretically, the best results are to be obtained from any combination of this character where the entire compression is taken by the concrete, and where the metal employed is so placed as to occupy the position of the extreme fiber on the tension side. Theoretically, it is also necessary, in order to develop the full strength of the metal, that it should be held in place by some means possessing greater strength than simple cohesion between the concrete and the steel. The difficulties to be encountered, in making any series of tests for the purpose of determining the laws governing the resistance of such combinations, were known by the fact that no formula had ever been suggested which even approximately accounted for the extremely high resistance of sections put in place in buildings and tested under concentrated loads. The companies interested in the manufacture of the materials used determined on having a series of tests made which should be devoted simply to the ascertaining of facts whether favorable or unfavorable, and the author was intrusted with the execution of these tests. The results obtained seem to be of sufficient value to justify their presentation to the Society, in order that they may be fully discussed and the real value of the combination may be generally known to engineers.

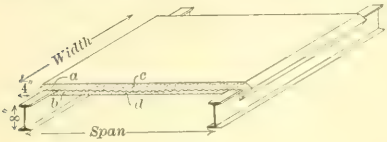


Fig. 1.

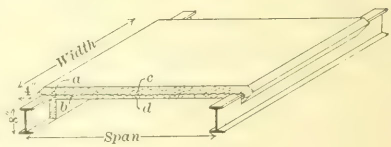


Fig. 2.

Slabs.—Fifty-six slabs of concrete and expanded metal of varying spans and thicknesses, both of concrete and of metal, but of practically uniform width, were erected on I-beam supports, of types shown in Figs. 1, 2, 3 and 4, and designated by the letters *A*, *B*, *C* and *D* in Table No. 1. These slabs were entirely independent of all others, so that the behavior of the material in each could be independently observed.

Cement.—The cements used were an American Portland and a slag cement. For the first of these the manufacturers claim a fineness of 90% on a No. 100 sieve, and 70% on a No. 200 sieve, and which actu-

ally showed a fineness of 96% on a No. 100 sieve, and 82% on a No. 180 sieve. A minimum tensile strength of neat cement, for seven days, of 400 lbs., and for 3 parts of sand to 1 part of cement, seven days, 150 lbs. was guaranteed. The tensile strengths under actual test exceeded the guaranteed amount considerably, the average of 30 tests giving 624 lbs. The slag cement is claimed to be ground to a fineness of 97% on a No. 180 sieve, and showed a fineness of 99.5% on a No. 100 sieve, and 98% on a No. 180 sieve. For neat cement, seven days, a tensile strength of 400 lbs. was claimed; for 3 parts of sand to 1 part of cement, seven days, 147 lbs. The slag cement ran very slightly below the amount claimed for neat cement. The fineness of the grinding, however, is a matter of very great importance, as will be shown later on.

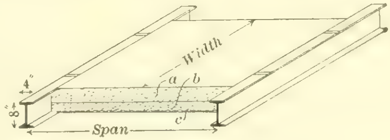


FIG. 3.

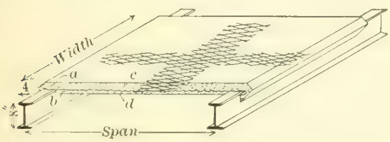


FIG. 4.

Sand.—Cow Bay sand, clean and sharp, unscreened, and of varying sizes from moderately fine sand up to some pebbles the size of a white bean, was used.

Cinders.—Ordinary steam cinders, varying in size from dust to pieces which would go through a $\frac{3}{4}$-in. ring, were used. Larger pieces were smashed by the shovel. The cinders were delivered by the barge-load for use in an adjoining building.

Stone.—The stone used was trap, broken to pass through a $1\frac{1}{2}$-in. ring, and practically uniform in size. It was too large to give the best results for stone concrete, but was the only stone available.

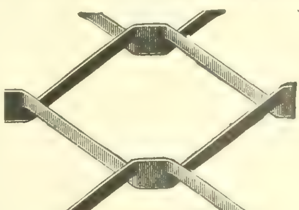


FIG. 5.

Gravel.—The gravel used was clean washed, running in size from $\frac{1}{2}$ in. to $1\frac{1}{2}$ ins.

Metal.—The metal used was known as expanded metal (Fig. 5) made from high grade, low-carbon Bessemer steel, containing 0.008 carbon; unannealed specimens, tested in tension parallel with the grain, showing a tensile strength of about 65 000 lbs., an elongation of about 15% and an elastic limit of 30 000 lbs. Tests made on strands of expanded metal, after having been subjected

to strain in the floor sections, showed an average ultimate strength of 54 240 lbs. The expanding is performed by partially shearing through the sheet and extending the metal about 11 per cent. The strands which broke in the work in all cases broke at a point where the metal was bent to secure the diamond shape of the mesh. It is commercially designated by giving the gauge of the steel and the amount of displacement between the junction of the meshes, thus : No. 10, 3-in. mesh, designates an expanded metal made from No. 10 steel in which the displacement of the bridge amounts to 3 ins., the axes of the diamond being $2\frac{3}{4}$ ins. and 6 ins. When expanded, this size and mesh weighs 0.56 lb. per square foot and has a sectional area per foot of width of 0.168 sq. in. and per inch of width of 0.014 sq. in. If the elastic limit then be taken at 30 000 lbs., the safe working strength per inch of width would be 200 lbs.

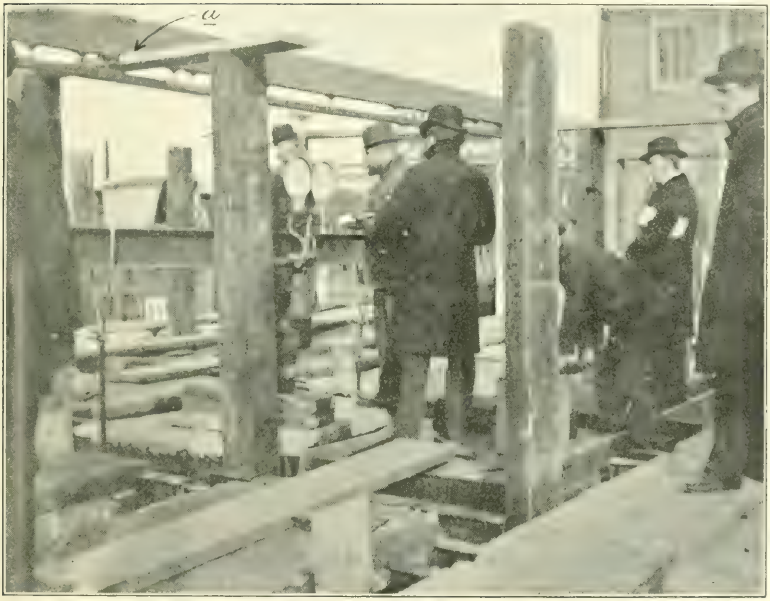
Expanded metal No. 4, 6-in. mesh, is made of No. 4 steel, in which the meshes are 5 ins. x 12 ins., its weight is 0.94 lb. per square foot, which equals 0.282 sq. in. of cross-section per foot of width, corresponding to 0.023 sq. in. per inch of width, or a working strength of 330 lbs. per inch of width.

In building the slabs, the expanded metal was usually put down in single sheets, with the long axis of the mesh at right angles to the beams, and so set as to be well covered by the concrete in the bottom of the slab. The sheets were sometimes lapped and sometimes doubled, and, when this was done, the only tying was that afforded by the concrete.

Testing Machine.—The testing machine was that employed and described in a paper previously submitted by the author,* the only addition being that there were two gauges, both of the same range, one new and just calibrated, and the other the old one, but they checked together exactly. The plunger was in excellent condition, and practically without friction except at high loads, where the friction loss would be, in any event, inappreciable. The machine was handled by means of a detachable track in two parts, furnished by the McCabe Manufacturing Company, one section of the track always being in advance of the testing machine. The holding down was accomplished by means of two cast-steel wire rope straps passing around cross beams which ran underneath the supporting beams. The general arrangement is shown in Plate V.

* *Transactions*, *Am. Soc. C. E.*, Vol. xxxiv, p. 544.

PLATE V.
PAPERS AM. SOC. C. E.
MARCH, 1898.
HILL ON STEEL CONCRETE CONSTRUCTION.



Loading.—Except in a few isolated cases, the loads were applied by means of a 3-in. x 12-in. piece of yellow pine 3 ft. long, placed in the center of the slab, with blocking between it and the plunger. This load amounted to about 100 lbs., and was not allowed for in computing the loads resisted by the slabs in any case. As the loading was increased, deflections were measured by means of a rule held by an assistant stationed beneath the slab. The conditions of loading were practically that of a load concentrated over the entire width of the slab in the center, in such a manner as to avoid local injury to the concrete.

Mixing.—All the concrete was mixed on a board platform by hand. The proper amount of sand was first dumped on the platform, then the cement was placed on it and the two mixed dry by turning over once with shovels, then the stone, gravel or cinders was added and the whole mass turned over once while dry with shovels, and turned into a heap. It was then raked down from the heap into a bed, and the water sprinkled on. It was turned over by shovels as the water was added, and again shoveled into a heap. From the heap it was shoveled into a wheel-barrow, wheeled to the slab, dumped and spread. It was then well rammed with an ordinary 8-lb. rammer, and was usually brought up to a smooth surface. All the mixing was done by laborers who had been employed on similar work.

Each mixture usually made two slabs and a cylinder approximately 8 ins. in diameter and 15 ins. long. After the slabs were formed, and before there was any frost, they were covered with 2 ins. of sand. Before the testing began, the sand was swept off and the centers removed. The cinder concrete weighed 100 lbs. per cubic foot, and the stone and gravel concrete 140 lbs. per cubic foot, as an average.

Weather.—Mixing began on November 13th, and testing began on December 16th. During the interval the minimum temperature was 21° Fahr., on November 24th, and the maximum was 66 degrees. The days when the temperature went below the freezing point were, on November 18th, 30°; 23d, 23°; 24th, 21°; 25th, 32°; 28th, 26°, and on the 30th, 25°, on which day it froze all day long; December 1st, 27°; 2d, 28°; 3d, 29°, and 4th, 29 degrees. From these records it will be seen that the slabs were unhurt by frost, there being sufficient time before freezing weather for the cement to set.

General.—The conditions were such as to produce a slab of average manufacture, in which the action of both of the materials under test could be minutely observed.

Cylinders.—The cylinders before mentioned were made for the purpose of determining the actual compressive strength of particular mixtures of concrete employed, so that, used in conjunction with the known tensile strength of the metal and the known breaking strength of the combination, it would be practicable to apply various formulas for the purpose of comparison. A number of them were broken in the testing machine heretofore described, and the remainder in the testing machine of the New York University Laboratory, the use of which was kindly tendered by Professor Collins P. Bliss. The results are given in Table No. 1.

TEST NO. 13; LOADS AND DEFLECTIONS.

FIG. 6.

Tests.—The results of the general tests of the slabs are summarized in Table No. 1 in all cases except for Tests Nos. 57 to 60. The curve of loading was similar in its character to that shown in Fig. 6, which is the curve for Test No. 13.

Test No. 57.—This was a test of three special light beams and concrete, being as shown in Fig. 7. The expanded metal was laid in widths of 18 ins., measured in a direction parallel with the beams, and lapped 4 ins. It was of No. 10 metal, and the number of square inches in a direction at right angles to the tensile strains would be 1.33. The major axis of the diamond, however, was at right angles

with the line of strain, and, in consequence, the effective resistance of this metal diminished in the ratio of 2.2 to 1.

The composition of the concrete was 1 part American Portland, 2 parts sand, and 5 parts cinders. The load was applied to 1 sq. ft. in the center of the span immediately over the flanges of the central beam. The ratio of span to depth was such that a weight of 150 lbs. produced a deflection of $\frac{1}{4}$ in., and 3 000 lbs. produced a deflection of $\frac{1}{2}$ in. At 5 500 lbs. the deflection was $\frac{15}{16}$ in., and the side beams began to separate from the concrete. At 6 500 lbs. the top flanges of the center beam buckled under the load and a deflection of $1\frac{7}{16}$ ins. was observed. This was the maximum strength. After the maximum

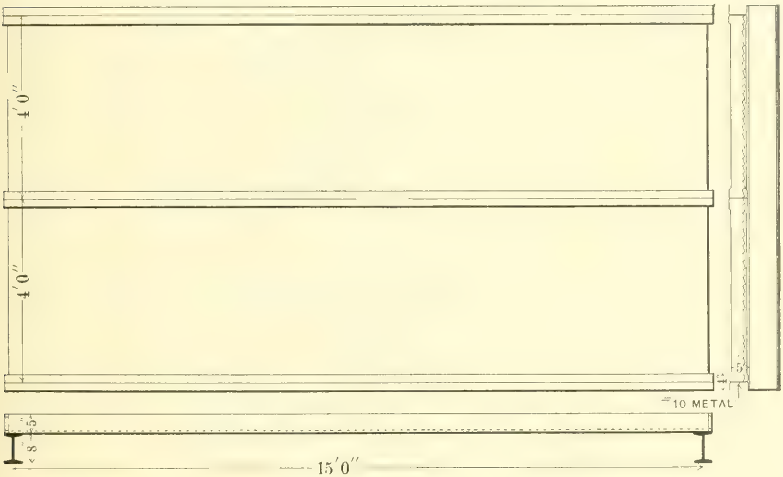


FIG. 7.

of 6 500 lbs. was reached, the increase in deflection and loss of resistance were very slow and uniform, reaching $3\frac{1}{8}$ ins., with a load of 5 800 lbs.

Test No. 58.—This was a test of a rectangular slab of concrete, 5 ins. thick, as shown in Fig. 8. The concrete was composed of American Portland, 1 part; sand, 2 parts, and cinders 5 parts. The metal was No. 10, in six pieces, and lapped 8 ins. longitudinally, and 20 ins. transversely; the total number of square inches of sectional area being 2.08. The load was applied to 1 sq. ft. in the center and was gradually increased. The elastic limit was at 10 500 lbs., and the deflection $\frac{11}{16}$ in. At 11 200 lbs., with a deflection of $\frac{13}{16}$ in., the corners of the slab began to lift,

and this lifting reached a maximum of $\frac{1}{4}$ in. at the corners, with a deflection of 2 ins. and a load of 11 000 lbs. While this pressure was maintained, a weight of 70 lbs. was twice dropped from a height of 2 ft. 4 ins. on the slab, immediately adjoining the loaded area, without

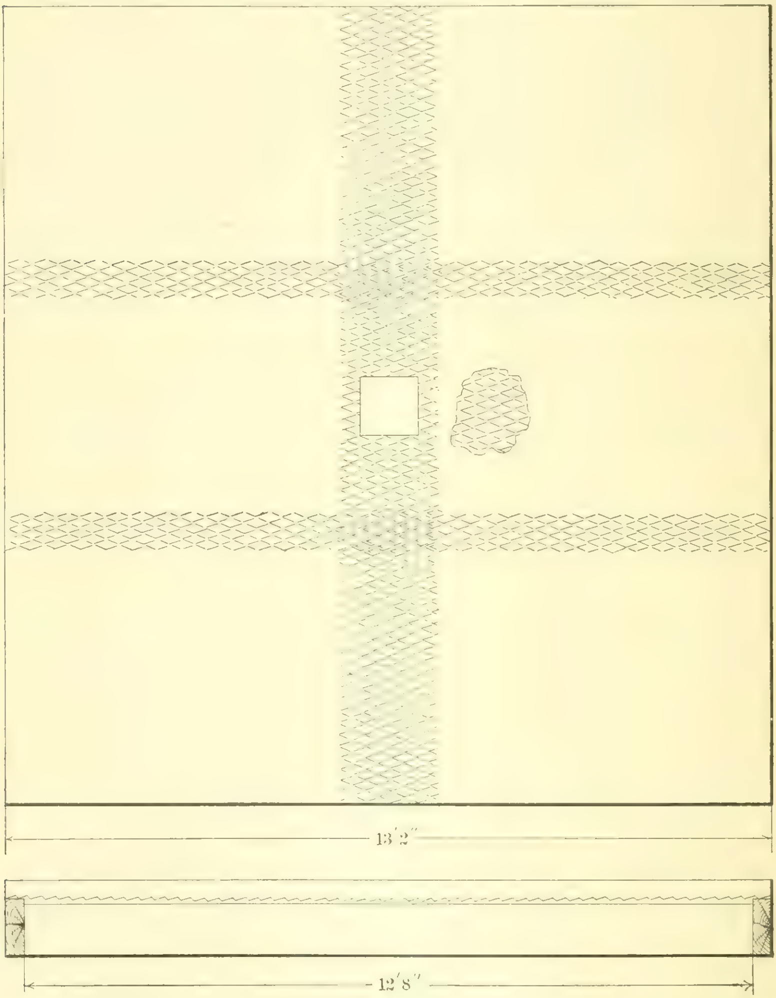


FIG. 8.

effect. A block $\frac{3}{4}$ sq. ft. in area was placed immediately adjoining the loaded area and a weight of 894 lbs. was dropped 1 ft. and decreased the pressure about 1 000 lbs., then it was twice dropped a distance of

15 ins., and twice a distance of 18 ins., the last time shearing an irregular hole of about 1½ sq. ft. in area, through the concrete, and decreasing the resistance to the plunger so that the load was in the vicinity of 4 500 lbs.

Test No. 59.—This was a test of a finished section of floor in the sugar house adjoining, and is shown in Fig. 9. The concrete was mixed in the proportion of American Portland cement, 1; sand, 2; cinders, 5. The top 2 ins. of the surface was mixed in the proportions of American Portland cement, 1; ¼-in. broken stone, 1, and sand, 1, and had a very smooth and handsome surface. The load was applied on an area of 1 sq. ft. The elastic limit was passed at a pressure of 35 000 lbs. The first crack appeared on the surface at a pressure of 37 600 lbs.; the maximum was reached with 37 750 lbs. The deflection was about 1½ ins., and then the failure became pronounced, cracks appearing in the top surface and roughly following the arc of a circle about 2 ft. distant from the point of application of the load for about one-half the area loaded, and, in addition, one crack running off diagonally at an angle of about 45° from the supporting beams. There were only slight cracks on the under surface.

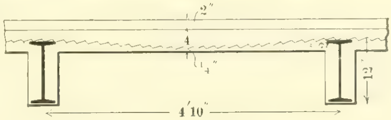


FIG. 9.

Test No. 60.—The section tested is shown in Fig. 10. Double expanded metal No. 10 was used. The concrete was the same as that used for Test No. 59. The load was applied on 1 sq. ft., and the arch resisted a total of 42 500 lbs. without sign of failure; the elastic limit was passed at about 35 000 lbs.

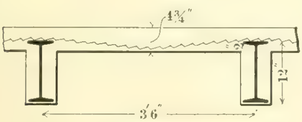


FIG. 10.

Observations.—The application of the load, in each case, was at a uniform rate. The deflections were practically at a uniform rate and were proportionate to the load up to the point where the elastic limit was reached. After the elastic limit was passed, they increased considerably; the bottom surface of the concrete began to show the compressing action of the strands of metal by breaking into small cracks and puckering up. Finally, the deflection became so great that small sections of the concrete between the strands fell off, and the strands of metal began to break. After the load curve showed a decided drop, the

pressure was relieved and usually the deflection decreased, the slabs showing some remaining elasticity. In no case was there any collapse of the section until ample warning had been given, and in no case was there a collapse of the section, except where the ratio of thickness of slab to span was far less than would ever be thought of in practice. In all cases the failure was identical in its character with that of an elastic slab supported at the ends and loaded in the center. The first crack, in almost every case, developed over the inner edge of the I-beam supporting the slab. The center of the slab usually crushed underneath the loading plank, but not at the edges of the plank. The bottom cracked across transversely in the center.

When the concrete was extremely hard, as in the stone and gravel mixtures, the slab did not curve between the point of application of the load and the point of support. In the cinder mixture there was sometimes a small but appreciable curvature. The supporting I-beams were steadied on top of the timbers with a couple of light spikes. In two or three cases there was a very slight rotation observed in the case of the soft concrete. In the remaining cases there was no movement of the beams. In some of the very long spans made with stone or gravel concrete the slabs rotated about the inner edge of the supporting beam flange and lifted at the outer edge of the flange an appreciable amount; in one case as much as $\frac{1}{4}$ in.

The general run of tests and the irregularity which exists among them, indicate the necessity of some more uniform method of mixing the concrete, if close calculations of the strength are to be made. The strengths shown by the mixtures in which the slag cement was employed are worthy of note, since the neat tensile strength tests showed a less strength than those in which American Portland was used, while in the actual use of the cement this difference disappears, and seems to indicate a considerable advantage due to fineness of grinding.

The tests were usually made in pairs, this being evident from a study of the tables, thus 1 and 2, 3 and 4, 5 and 6, etc., were made from the same mixtures of concrete and at the same time. It will be observed that the two slabs from the same mixture at times vary as much as two slabs made in the same proportions, but of different mixtures. It will be further observed that, in the case of the pair 5–6, in one case the metal was broken and the other not. The same thing is true of the pairs 7–8, 9–10, 17–18, 36–37, 40–41 and 44–45, thus indicat-

ing that the proportions of strength of concrete to strength of steel for this particular form of loading were practically correct, as in one case the steel and in the other case the concrete failed first. This is further emphasized by the tests 15–16, 31 and 33, in which there was an excess of metal and consequently no complete failure thereof; also by the tests 25–26, in which the compressive strength of the cinder concrete was exceptionally low and therefore the metal was not broken.

Constants. —Taking up the tests which seem to be fairly uniform, it is the author's judgment that a fair value for the ultimate compressive strength of a fairly well-mixed cinder concrete in the proportion of 1–2–5, or a thoroughly well mixed cinder concrete in the proportion of 1–3–6, should be 400 lbs. per square inch. A stone or gravel concrete made of graded stones and thoroughly mixed by machinery should show a strength of at least 800 lbs. per square inch; in each case the strengths being for concretes 28 days old. The tensile strength of the concrete may be taken to be one-fifth of the compressive strength. In Table No. 2 is given a reduction of certain tests on cinder concrete slabs to a uniform condition of span, width and depth. Two points are noted for each test, first, that at which the deflection reached a point sufficiently great to cause a cracking in the plaster when applied to the under side; second, that at which the elastic limit was passed.

These tests show that a centrally applied load of 3 200 lbs. produces a deflection sufficient to crack plaster; a compressive strain of 300 lbs. per square inch in the concrete, and a tensile strain in the metal of 322 lbs. per inch of width, and a load of 4 580 lbs., reached the elastic limit of the combination, producing a compressive strain of 400 lbs. per square inch in the concrete, and a tensile strain of 500 lbs. per inch of width in the metal.

These results are obtained by the application of the formula hereinafter noted, on the assumption that the concrete and the steel will be strained proportionately within the elastic limit and up to the maximum at the elastic limit, and that if there is any reserve of strength it will exist only in the steel.

It is essential that in no case should the deflection produced by a load be sufficiently great to cause a cracking, either in the plaster or in the floor itself. It can also be seen from an examination of Table No. 1 that if the plaster does not crack, the concrete will not, since the deflection corresponding to the first crack is, in almost all cases, three

times as great as that required to crack the plaster. And, finally, it must be borne in mind that commercial conditions require the use of a cement so good that the contractor can remove his centers within a week or ten days, and in consequence any slab which will then bear its own weight is absolutely proof against collapse at any future time. Keeping these facts in view and considering that it is well to be conservative in all cases in the use of a new material, the author has used as his constants, for safe working strains of cinder concrete in compression 75 lbs. per square inch, and for stone concrete in compression 150 lbs. per square inch, which would correspond to a deflection under uniformly distributed loads of about one-quarter of that required to crack plaster, and to strains of about one-fifth of the elastic limit of the concrete.

Notation.—The following notation has been employed in the development of the formula:

$C =$ Safe working strength of concrete per square inch, or 75 lbs. for cinders and 150 lbs. for stone.

$T =$ Safe working strength of concrete in tension per square inch $= \frac{C}{5}$.

$S =$ Safe working strength of steel per inch of width, or 200 lbs. for No. 10 metal, and 330 lbs. for No. 4 metal.

$M =$ Bending moment of the external forces in inch-pounds.

$R =$ Resisting moment of the section in inch-pounds.

$l =$ Span in inches.

$t =$ Thickness of slab or section in inches.

$x =$ Distance from edge of slab or section to the neutral axis on the compression side.

$y =$ Distance from the edge of slab or section to the neutral axis on the tension side.

$W =$ Safe uniformly distributed load per square foot.

$w =$ Safe uniformly distributed load per square inch.

Then when $W = 75$, $w = 0.52$;
$W = 125$, $w = 0.87$;
$W = 175$, $w = 1.22$;
$W = 250$, $w = 1.74$.

Formula.—The formula may be developed within the elastic limit of the material as follows:

Taking a section, it is evident that the compression in the concrete above the neutral axis must equal the tension below the neutral axis

due to the action of both the concrete and the steel. Taking any section of unit width and adopting the notation given, the compression is represented by $\frac{C x^2}{2}$. The tension will be represented by $\frac{T y^2}{2} + S y$.

These two combined give $R = \frac{C x^2}{2} + \frac{T y^2}{2} + S y$ ......(Formula 1.)

It is evident that $R = C x^2$ ....(Formula 2.)

These are strictly correct when y is the distance from the neutral axis to the center of the expanded metal. They are within an extremely small percentage of being correct when the metal is imbedded in the bottom of the concrete, and the result obtained by the formula will be practically correct and absolutely safe if the thickness of the slab be made $\frac{1}{4}$ in. in excess of the thickness determined by computation. Slabs may be built of any of the styles shown in Figs. 11, 12 and 13. In Fig. 11 the floor slab is in the condition of a continuous girder, sup-

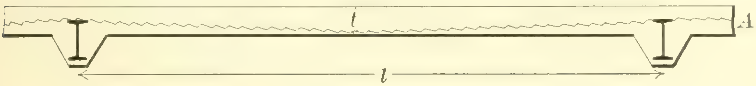


FIG. 11.

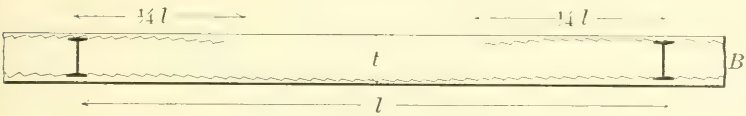


FIG. 12.

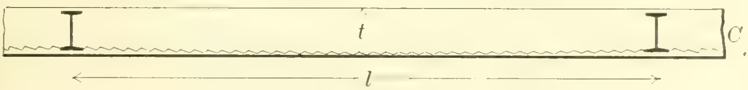


FIG. 13.

ported at intervals. The moment of resistance of the section at the center is given by Formula 1. Over the points of support, since there is no steel, the moment may be expressed, with practical accuracy, by

$$R = \frac{C x^2}{2} + \frac{T y^2}{2} = 0.47 \ T t^2 \text{ ....(Formula 3.)}$$

In Fig. 12 a condition exists similar to that of Fig. 11, except that, owing to the position of the concrete, there is no tensile strength to be developed from it in the section over the point of support, and consequently Formula 1 is modified thus:

$$R = \frac{C x^2}{2} + S y \text{ ....(Formula 4.)}$$

For conditions of uniformly distributed load and unit width, the moment of the external forces and the resistances of the sections in the center for the various cases is represented by:

$$M = \frac{w\,l^2}{8} = \frac{C\,x^2}{2} + \frac{T\,y^2}{2} + S\,y = R \ldots\ldots \text{(Formula 5.)}$$

$$M = \frac{w\,l^2}{24} = \frac{C\,x^2}{2} + \frac{T\,y^2}{2} + S\,y = R \ldots\ldots \text{(Formula 6.)}$$

and the sections over the points of support are represented by

$$M = \frac{w\,l^2}{12} = 0.47\ T\,t^2 = R \ldots\ldots\ldots\ldots\ldots\ldots \text{(Formula 7.)}$$

$$M = \frac{w\,l^2}{12} = \frac{C\,x^2}{2} + S\,y = C\,x^2 = R \ldots\ldots \text{(Formula 8.)}$$

VALUES OF x AND y, IN FORMULA 1.

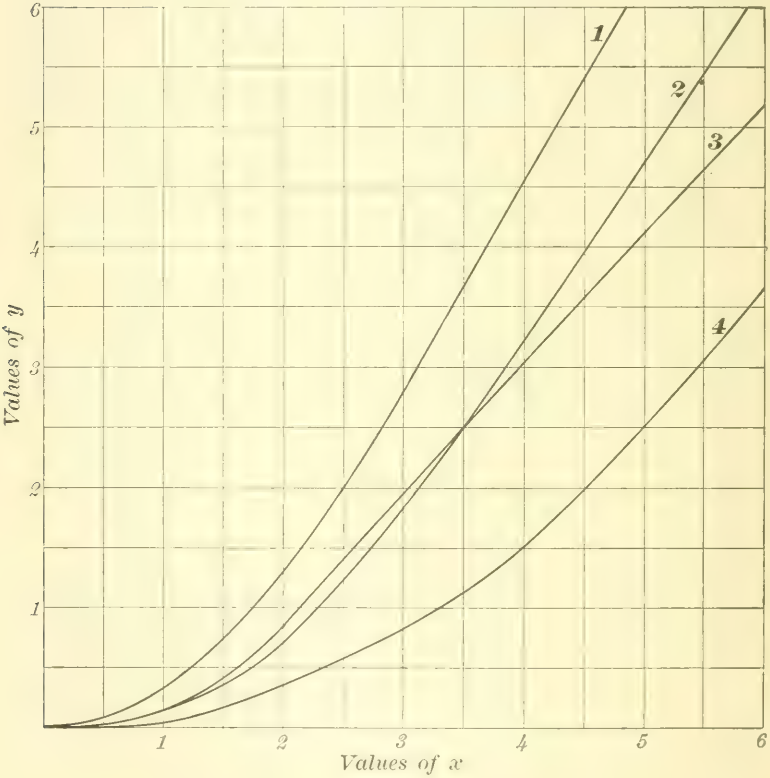


FIG. 14.*

In the practical application it is well to remember that in all cases $\frac{C\,x^2}{2}$ represents half the total resistance.

Using this in conjunction with the other formulas given, the curves in the diagrams shown in Figs. 14 and 15 have been calculated. These

* Fig. 14 applies to center sections.
Curve 1—Stone concrete and No. 10 metal.
" 2— " " " No. 4 "
" 3—Cinder " " No. 10 "
" 4— " " " No. 4 "

curves gives the values of x and y. It is therefore feasible to substitute M for R in Formula 2, and having obtained x find the appropriate value of y for any of the conditions given. In this way the tables of requirements for safe uniformly distributed loads have been computed. It is to be particularly observed that no formulas are given for the resistance to concentrated loads. This is for the reason that concentrated loads so rarely occur, and the resistance to them is so great

VALUES OF x AND y, IN FORMULA 8.

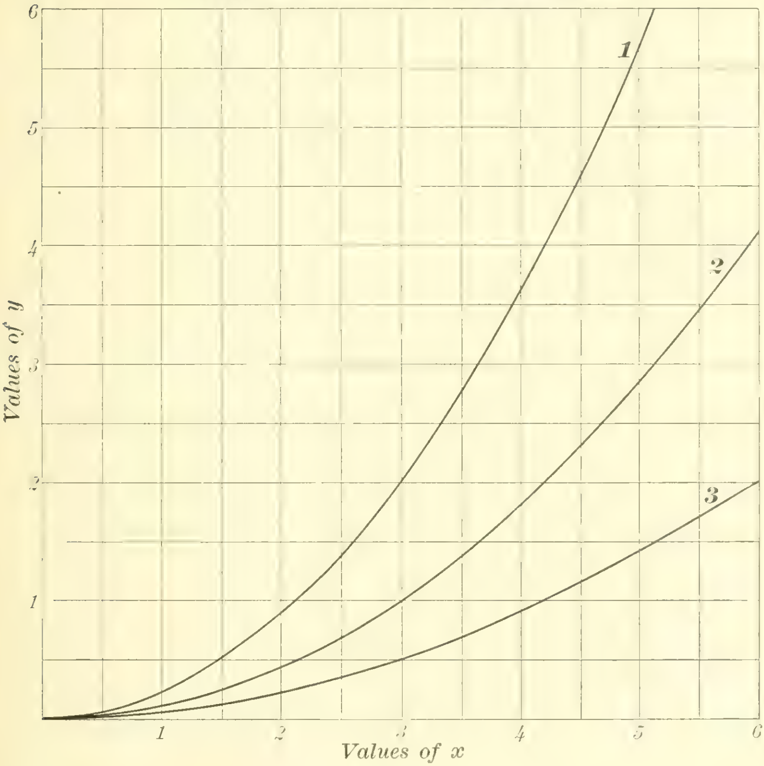


FIG. 15.*

as to render it practically a waste of time. This resistance may be calculated on the assumption that the resistance will equal that of a circular disc, of a diameter equal to the span, fixed on all edges and loaded in the center.

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* Fig. 15 applies to sections over the point of support.
Curve 1—Stone concrete and No. 4 metal.
" 2 { " " " No. 4 " doubled.
  { Cinder " " No. 4 "
" 3— " " " No. 4 " doubled.

TABLE No. 1.

Test number.	Age, in days.	Span.	Width.	*a.*	*b.*	*c.*	*d.*	Square inches of metal in cross-section.	Load causing deflection that would crack plaster.	Deflection.	Elastic limit.	Deflection.	Load producing first crack.	Deflection.	Load at which metal broke.	Deflection.	Maximum load.	Deflection.
1	18	60	42.0	....	....	3.25	....	0.58	......	....	....	....	....	....	....	....	2.8	4.0
2	31	60	42.5	1.88	1.50	3.50	0.38	0.58	1.50	0.16	2.5	0.62	2.4	0.54	not	....	3.0	0.5
3	31	60	42.75	1.65	1.31	2.88	0.75	0.58	3.0	0.16	3.4	0.31	3.25	0.25	3.50	1.50	4.1	0.69
4	31	60	42.0	2.00	1.69	3.25	0.75	0.58	2.75	0.16	5.0	0.81	4.00	0.37	5.00	1.00	5.0	1.00
5	30	60	42.0	1.94	1.50	3.45	0.05	0.58	2.10	0.16	2.5	0.33	2.90	0.50	not	....	3.1	1.00
6	31	60	42.0	1.87	1.31	3.30	0.20	0.58	2.5	0.16	3.0	0.37	2.9	0.31	3.0	1.50	3.4	1.06
7	30	60	41.5	2.0	1.4	3.50	0.25	0.58	2.6	0.16	3.4	0.31	3.5	0.37	3.6	3.50	3.6	3.00
8	30	60	42.5	1.85	1.69	3.50	0.25	0.58	2.3	0.16	3.0	0.56	3.0	0.69	not	....	3.0	1.5
9	30	60	41.0	1.88	1.31	3.50	0.12	0.58	3.5	0.16	4.0	0.28	4.2	0.50	3.5	2.0	4.2	0.44
10	30	60	42.0	2.06	1.44	3.25	0.50	0.58	3.8	0.16	4.6	0.50	4.6	0.50	not	....	4.6	0.50
11	25	60	42.0	1.88	1.31	3.75	0.06	0.58	5.0	0.16	5.2	0.22	6.05	0.37	4.0	0.94	6.05	0.37
12	25	60	42.0	2.00	1.44	4.44	0.06	0.58	3.5	0.16	4.0	0.37	4.2	0.50	4.25	2.75	4.4	2.0
13	28	60	42.5	2.00	1.31	3.75	0.25	0.97	4.1	0.16	5.6	0.75	5.5	2.25	5.5	2.25	6.15	0.88
14	28	60	43.0	1.94	1.75	4.38	0.12	0.97	4.1	0.16	5.5	0.50	5.55	0.88	5.9	1.31	6.0	1.81
15	28	60	42.0	1.81	1.75	4.75	0.0	1.16	5.1	0.16	9.3	0.69	9.3	0.69	not	....	9.3	0.69
16	28	60	42.0	1.75	1.50	3.94	0.0	1.16	5.6	0.16	9.3	0.62	....	....	not	....	9.8	1.50
17	30	60	42.0	1.81	1.62	3.80	0.07	0.58	3.9	0.16	4.4	0.25	4.5	0.38	not	....	4.8	0.75
18	30	60	42.0	1.94	1.50	3.50	0.50	0.58	4.9	0.16	6.5	0.38	6.5	0.38	5.8	1.31	6.5	0.50
19	30	60	43.0	4.25	3.68	0.06	....	0.58	2.4	0.16	5.0	0.56	....	....	5.0	0.56	5.0	0.56
20	30	60	42.0	4.75	3.0	0.25	....	0.58	2.9	0.16	5.0	0.75	5.1	1.25	5.0	2.38	5.10	1.25
21	30	60	41.0	4.0	3.87	0.12	....	0.58	3.1	0.16	4.5	0.37	4.7	0.56	4.30	2.25	4.85	1.0
22	30	60	42.0	4.0	3.87	0.12	....	0.58	3.4	0.16	5.48	0.50	5.48	0.50	5.0	1.75	5.6	0.75
23	33	60	40.0	4.0	3.75	0.25	....	0.58	4.4	0.16	7.9	0.44	....	....	8.6	1.12	8.6	0.88
24	33	60	40.5	4.25	3.25	0.50	....	0.58	3.4	0.16	6.0	0.56	....	....	8.0	2.00	8.30	2.50
25	33	60	41.0	4.37	3.25	0.37	....	0.58	2.9	0.16	4.5	0.75	....	....	not	....	5.60	2.62
26	33	60	42.5	4.00	3.88	0.12	....	0.58	3.1	0.16	6.0	0.75	....	....	not	....	7.2	2.50
27	32	72	42.0	2.12	1.12	4.12	0.12	0.60	3.0	0.20	3.9	0.50	....	....	3.8	0.31	3.95	0.94
28	32	72	42.0	2.00	1.19	3.68	0.25	0.60	2.5	0.20	3.9	0.62	4.0	0.88	3.2	1.81	4.0	0.88

TABLE No. 1—(*Continued*).

REMARKS.—All loads in thousands of pounds. All dimensions and deflections in inches.
Tests 1 to 33, both inclusive, made with American Portland cement.
Tests 34 to 56, both inclusive, made with slag cement.

Test number.	Final load.	Deflection.	Permanent deflection.	Type of slab.	Cylinders. Diameter.	Square inches, sect. area.	Total load.	Load per square inch.	Length.	Remarks.
1	....	....	....	*A*	8.48	56.7	16.5	292	......	Cinders, 1–3–6.
2	2.9	2.00	....	*A*	....	....	......	....	......	Cinders, 1–3–6. Good top surface, mixed very moist.
3	3.2	2	....	*A*	....	....	......	....	......	Cinders, 1–2–5. Mixed very moist.
4	3.0	3	....	*A*	....	....	......	....	......	Cinders, 1–2–5. Mixed very moist. Small section of bottom near center and below metal fell out.
5	2.0	2	....	*A*	8.53	57.2	22.7	398	16.0	Stone, 1–3–6. Mixed a little too dry. N. edge sheared down. Top crushed in.
6	2.7	2	....	*A*	....	....	......	....	......	Stone, 1–3–6. Mixed about right. Top crushed. Bottom opened in one crack.
7	3.1	4	....	*A*	8.38	55.2	34.5	625	......	Stone and gravel graded, 1–2–10. Concrete moist. Top surface rough. Top crushed, bottom spaced and cracked.
8	2.5	3	....	*A*	....	....	......	....	......	Stone and gravel graded, 1–2–10. Concrete moist. Top surface rough. Top crushed, bottom spaced and cracked.
9	3.5	2	....	*A*	8.48	56.7	52.0	917	......	Gravel, 1–3–6. Concrete a little wet. Fine top surface. Crushed, bottom cracked.
10	4.0	2	....	*A*	....	....	......	....	......	Gravel, 1–3–6. Concrete a little wet. Fine top surface. Crushed, bottom cracked.
11	4.0	0.94	....	*A*	8.17	52.3	46.0	880	12.25	Gravel, 1–2–7. Fine top surface. Crushed 15 ins. N. of center on diagonal line.
12	3.75	2.81	2.0	*A*	....	....	......	....	......	Gravel, 1–2–7. Good surface. Section of bottom below metal dropped out.
13	5.0	2.75	2.25	*B*	....	....	......	....	......	Cinders, 1–2–5. Good surface. Top crushed. Edges lifted, bottom cracked.
14	5.8	2.31	1.62	*B*	....	....	......	....	......	Cinders, 1–2–5. Good surface. Top crushed. Edges lifted, bottom cracked.
15	9.0	1.06	0.75	*B*	....	....	......	....	......	Gravel, 1–3–6. Fair surface. Top cracked diagonally.
16	9.5	2.31	1.31	*B*	....	....	......	....	......	Gravel, 1–3–6. Fair surface. Top crushed near center.
17	3.8	2.56	2.06	*B*	....	....	......	....	......	Stone, 1–2.4–4.1. Fair surface. Top crushed. Cement fell off bottom below metal.
18	5.0	2.38	2.0	*B*	....	....	......	....	......	Stone, 1–2.4–4.1. Fair surface. Top crushed. Cement fell off bottom below metal.
19	3.5	1.12	1.12	*C*	8.47	56.3	28.35	503	......	Stone, 1–2.4–4.1. Filling of Rosendale, 1; sand, 2; cinders, 10; about twelve days old.
20	4.5	4.5	4.12	*C*	....	....	......	....	......	Stone, 1–2.4–4.1. Filling of Rosendale, 1; sand, 2; cinders, 10; about twelve days old.
21	4.0	3.25	2.88	*C*	....	....	......	....	......	Gravel, 1–3–6. Filling like Test 19. Loaded on 1 sq. ft. Concrete fell from bottom.
22	4.0	2.0	1.75	*C*	....	....	......	....	......	Gravel, 1–3–6. Filling like Test 19.
23	6.6	1.44	1.12	*C*	8.31	54.2	16.5	305	......	Cinders, 1–2–5. Filling like Test 19. Filling somewhat frozen.
24	7.5	3.00	2.56	*C*	....	....	......	....	......	Cinders, 1–2–5. Filling like Test 19. Filling somewhat frozen.
25	5.0	4.25	....	*C*	8.38	55.2	13.53	246	15.5	Cinders, 1–3–6. Filling like Test 19. Bottom much scaled. Top not frozen.
26	6.0	3.62	3.25	*C*	....	....	......	....	......	Cinders, 1–3–6. Filling like Test 19. Bottom much scaled. Top not frozen.
27	2.50	1.25	0.69	*A*	8.31	54.2	26.5	490	......	Cinders, 1–2–5. Metal in two pieces, with longitudinal lap of 1½ ins.
28	2.5	2.18	1.81	*A*	....	....	......	....	......	Cinders, 1–2–5. Metal in two pieces, with longitudinal lap of 2 ins. Good surface, bottom cracked.

TABLE No. 1—(*Continued*).

Test number.	Age, in days.	Span.	Width.	*a.*	*b.*	*c.*	*d.*	Square inches of metal in cross-section.	Load causing deflection that would crack plaster.	Deflection.	Elastic limit.	Deflection.	Load producing first crack.	Deflection.	Load at which metal broke.	Deflection.	Maximum load.	Deflection.
29	33	96	42.0	2.12	1.19	4.25	0.13	0.65	1.8	0.27	2.3	0.50	....	....	2.5	1.06	2.5	1.06
30	33	96	41.5	2.06	1.13	4.26	0.25	0.66	1.8	0.27	2.4	0.62	....	....	2.4	0.62	2.4	0.62
31	32	120	42.0	2.06	1.31	4.26	0.25	1.38	2.1	0.31	3.5	1.00	....	....	not	....	3.7	1.50
32	32	108	41.0	2.06	1.19	4.44	0.12	0.65	2.4	0.30	3.0	0.56	....	....	3.0	2.0	3.5	1.00
33	33	120	42.0	2.19	1.38	4.13	0.37	1.94	2.70	0.30	4.0	1.00	4.25	1.38	not	....	4.3	1.50
34	24	60	42.5	2.56	1.12	3.70	0.12	0.58	3.0	0.16	4.5	0.75	3.4	0.25	4.65	1.69	4.65	1.69
35	24	60	42.0	2.69	1.19	3.25	0.50	0.58	2.7	0.16	5.25	0.81	4.5	0.44	4.7	3.25	5.25	0.81
36	30	60	42.0	1.81	1.25	3.37	0.12	0.58	3.0	0.16	3.7	0.50	3.0	0.19	3.8	3.25	4.25	2.37
37	30	60	42.5	1.81	1.25	3.12	0.37	0.58	2.6	0.16	5.0	0.75	3.5	0.37	not	....	5.05	1.0
38	30	60	42.5	1.88	1.56	3.7	0.05	0.58	3.6	0.16	5.95	0.75	5.0	0.33	5.5	1.12	5.95	0.75
39	30	60	43.0	1.81	1.12	3.25	0.25	0.58	2.6	0.16	4.5	0.75	4.0	0.44	4.3	1.69	4.5	1.50
40	30	60	42.0	1.94	1.31	2.94	0.87	0.58	3.1	0.16	5.2	0.50	4.5	0.31	4.75	1.25	5.2	0.75
41	30	60	42.0	1.81	1.50	3.31	0.75	0.58	3.6	0.16	6.0	0.50	6.0	0.62	not	....	6.0	0.50
42	31	60	42.0	1.88	1.25	3.81	0.25	0.58	3.1	0.16	5.5	0.50	5.6	0.56	5.0	0.38	5.6	0.56
43	31	60	42.0	1.88	1.31	2.88	0.50	0.58	3.2	0.16	4.5	0.44	0.45	0.50	4.0	1.25	4.6	1.00
44	31	60	42.0	1.69	1.56	2.94	0.87	0.58	3.1	0.16	4.75	0.75	3.9	0.33	not	....	4.90	0.81
45	31	60	43.0	2.0	1.56	3.69	0.25	0.58	4.6	0.16	5.40	0.25	5.4	0.25	5.5	2.5	5.7	2.0
46	30	60	45.5	4.5	3.25	0.25	....	0.58	3.6	0.16	5.6	0.75	5.5	0.44	4.6	3.5	5.75	1.00
47	30	60	43.0	4.37	3.37	0.25	....	0.58	4.6	0.16	5.8	0.50	5.5	0.28	6.1	0.87	6.1	0.87
48	30	60	41.0	4.50	3.25	0.25	....	0.58	2.6	0.16	5.0	0.75	....	....	5.4	1.87	5.4	1.62
49	30	60	43.5	4.50	3.0	0.50	....	0.58	2.9	0.16	6.0	0.61	....	....	5.5	2.75	6.2	1.50
50	31	72	42.5	2.00	1.38	4.12	0.25	0.58	3.3	0.20	4.9	0.50	....	....	4.5	2.25	5.0	0.62
51	31	72	42.5	2.00	1.25	4.32	0.12	0.58	3.5	0.20	4.5	0.44	....	....	4.5	1.50	4.60	1.0
52	32	96	43.0	2.00	1.12	3.88	0.25	0.59	2.1	0.27	2.5	0.30	....	....	2.5	1.25	3.0	0.62
53	28	96	42.5	2.12	1.12	3.38	0.37	0.65	2.1	0.27	2.9	0.50	2.9	0.5	2.5	1.00	2.9	0.50
54	28	120	42.0	2.12	1.25	3.0	0.25	0.61	1.3	0.30	2.0	1.00	1.8	0.75	1.5	2.75	2.15	1.25
55	28	120	42.0	2.00	1.06	4.06	0.13	0.64	1.2	0.30	1.9	1.00	1.9	2.0	1.6	2.25	2.0	1.5
56	33	120	42.0	2.19	1.37	3.87	0.06	1.16	2.5	0.30	4.2	0.88	4.5	1.0	4.5	2.0	4.75	1.50

TABLE No. 1—(*Continued*).

Test number.	Final load.	Deflection.	Permanent deflection.	Type of slab.	Cylinders. Diameter.	Cylinders. Square inches, sect. area.	Cylinders. Total load.	Cylinders. Load per square inch.	Cylinders. Length.	Remarks. All loads in thousands of pounds. All dimensions and deflections in inches. Tests 1 to 33, both inclusive, made with American Portland cement. Tests 34 to 56, both inclusive, made with slag cement.
29	....	....	...	*D*	8.54	57.2	26.5	464	......	Cinders, 1-2-5. Metal in four pieces, lapping 7½ ins. longitudinally and 1 ft. transversely. Good surface.
30	....	....	....	*D*	....	....	......	....	......	Cinders, 1-2-5. Metal in four pieces, lapped 6 ins. longitudinally and transversely.
31	3.1	2.56	1.62	*D*	....	....	......	....	......	Cinders, 1-2-5. Metal in eight pieces, doubled, lapped 6 ins. longitudinally, 18 ins. transversely. Good surface.
32	2.5	2.75	2.38	*D*	8.38	55.2	32.5	590	......	Cinders, 1-2.4-6. Metal in four pieces, lapped 6 ins. longitudinally, 18 ins. transversely. Load applied at two points, dividing span into three equal parts.
33	3.00	3.62	3.00	*D*	....	....	......	....	......	Stone, 1-3-5. Metal doubled in four pieces, lapped 18 ins. transversely. Good top surface, crushed.
34	4.0	2.5	....	*A*	....	....	......	....	......	This and all remaining tests made with slag cement. Cinders, 1-2.4-4.8. Good surface. Part of bottom below metal fell off.
35	4.5	3.5	....	*A*	....	....	......	....	......	Cinders, 1-2.4-4.8. Good surface. Load on 1 sq. ft. Top crushed, bottom cracked.
36	3.8	3.25	....	*A*	8.63	58.5	20.0	342	......	Cinders, 1-1.7-4.2. Bottom blistered. Top surface good and crushed. Metal strands drawn together.
37	3.3	3.5	....	*A*	....	....	......	....	......	Cinders, 1-1.7-4.2.
38	3.0	1.31	0.88	*A*	8.35	54.8	40.0	730	......	Stone, 1-2.4-4. Rough top surface.
39	3.8	3.0	1.56	*A*	....	....	......	....	......	Stone, 1-2.4-4. Top slightly crushed.
40	4.2	1.75	1.37	*A*	8.48	56.7	35.0	618	......	Gravel, 1-3-6. Concrete a little wet; fine top surface. Crushed on top, cracked below.
41	5.00	2.71	2.31	*A*	....	....	......	....	......	Gravel, 1-3-6. Concrete a little wet; fine top surface. Crushed on top, cracked below.
42	3.5	1.06	0.75	*A*	....	....	......	....	......	Gravel, 1-2.4-4.9. Fair top surface. Top crushed.
43	3.5	2.25	1.94	*A*	....	....	......	....	......	Gravel, 1-2.4-4.9. Fair top surface. Top crushed.
44	4.0	2.0	1.62	*A*	8.35	54.8	40.0	730	......	Gravel, 1-1.6-6.5. A little moist. Fair top surface.
45	5.0	2.66	2.31	*A*	....	....	......	....	......	Gravel, 1-1.6-6.5. A little moist. Fair top surface.
46	3.75	3.87	3.56	*C*	8.28	53.8	35.0	650	......	Gravel, 1-2.4-4.8. Filling like Test 19. Top crushed in.
47	5.3	1.87	1.61	*C*	....	....	......	....	......	Gravel, 1-2.4-4.8. Filling like Test 19. Top crushed in.
48	4.8	2.75	....	*C*	8.48	56.7	18.72	330	15.5	Cinders, 1-1.6-4. Filling like Test 19.
49	5.0	3.19	2.75	*C*	....	....	......	....	......	Cinders, 1-1.6-4. Filling like Test 19.
50	....	....	1.75	*A*	8.47	56.3	43.0	765	15.25	Cinders, 1-1.6-4.
51	3.9	2.94	1.56	*A*	8.47	56.3	43.0	765	15.25	Cinders, 1-1.6-4. Bottom cracked.
52	....	....	....	*A*	....	....	......	....	......	Cinders, 1-1.6-4. Good surface.
53	....	....	....	*D*	....	....	......	....	......	Cinders, 1-1.6-4. Good surface. Metal in four pieces, lapped, 14 ins. transversely, and 4 ins. longitudinally.
54	....	....	....	*D*	....	....	......	....	......	Cinders, 1-1.6-4. Good surface. Metal in four pieces, lapped 15 ins. transversely. Edges lifted.
55	1.0	3.0	....	*D*	....	....	......	....	......	Cinders, 1-1.6-4. Good surface. Metal in four pieces, lapped, 15 ins. transversely. Top crushed. Edges lifted.
56	4.00	2.44	1.75	*D*	8.35	54.8	35.27	645	......	Stone, 1-2.4-4. Metal doubled in four pieces, lapped 24 ins. transversely. Fair surface.

TABLE No. 2.

Test No.	Load producing deflection sufficient to crack plaster.	Elastic limit.		Test No.	Load producing deflection sufficient to crack plaster.	Elastic limit.	
3	3.64	4.13	....	37	2.90	....	5.60
4	2.95	5.40	....	48	3.10	....	5.40
13	3.82	5.20	....	49	3.40	....	7.00
14	3.30	4.40	....	50	3.40	5.00	....
27	3.10	4.00	....	51	3.40	4.40	....
28	2.85	3.80	....	52	3.00	....	3.60
34	2.80	4.30	....	53	3.50	4.80	....
35	2.90	....	5.60	54	3.10	4.70	....
36	3.10	3.90	....				
				Average.	3.20	4.58	5.44

Cinder concrete only. Loads in thousands of pounds.

Last column shows elastic limit, where conditions affected this limit, but not the plaster limit.

TABLE No. 3.—Characteristics of Floor Slabs for Carrying the Designated Live Loads. Safe Working Strength of Cinder Concrete 75 Lbs. in Compression, 15 Lbs. in Tension. Of Stone Concrete, 150 Lbs. and 30 Lbs., Respectively. Of No. 10 Metal, 200 Lbs. per Inch-Width. Of No. 4 Metal, 330 Lbs. per Inch-Width.

75 lbs. per square foot, live load, uniformly distributed.

Span	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Concrete	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*S*	*S*	*S*	*S*	*S*	*S*
Top	.....	.....	.....	.....	.....	4	4	4*D*	4*D*	4*D*	4*D*	4*D*	4*D*	4*D*	4*D*
Bottom	10	10	10	4	4	10	10	10	10	10	10	10	10	10	10
Thickness	2.75	3.5	5	4.75	5.5	5	6	5.75	6.5	5.5	6.0	6.75	7.5	8.5	9.25
Type	*A C*	*A C*	*A C*	*A C*	*A C*	*B*	*B*	*B*	*B*	*B*	*B*	*B*	*B*	*B*	*B*

125 lbs. per square foot, live load, uniformly distributed.

Span	3	4	5	6	7	8	9	10	11	12	13	14	15
Concrete	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*S*	*S*	*S*
Top	.....	.....	.....	.....	.....	4	4	4	4*D*	4*D*	4*D*	4*D*	4*D*
Bottom	10	10	10	4	4	10	10	10	10	10	10	10	10
Thickness	2.25	3	4.25	4	5	5	5.75	7	6.75	7.5	6.5	7.5	8
Type	*A C*	*A C*	*A C*	*A C*	*A C*	*B*	*B*	*B*	*B*	*B*	*B*	*B*	*B*

175 lbs. per square foot, live load, uniformly distributed.

Span	3	4	5	6	7	8	9	10	11	12	13	14	15
Concrete	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*C*	*S*	*S*	*S*	*S*	*S*
Top	.....	.....	.....	.....	.....	4	4	4*D*	4*D*	4*D*	4*D*	4*D*	4*D*
Bottom	10	10	10	4	4	10	10	10	10	10	10	10	10
Thickness	2.5	3.5	5.0	5	6.25	6	7	8	6	7	8	9	10
Type	*A C*	*A C*	*A C*	*A C*	*A C*	*B*	*B*	*B*	*B*	*B*	*B*	*B*	*B*

250 lbs. per square foot, live load, uniformly distributed.

Span	3	4	5	6	7	8	9	10	11	12
Concrete	*C*	*C*	*C*	*S*	*S*	*C*	*C*	*C*	*S*	*S*
Top	.....	.....	.....	.....	.....	4	4*D*	4*D*	4*D*	4*D*
Bottom	10	10	10	4	4	10	10	10	10	10
Thickness	2.75	3.75	4.75	5.25	8	7	7.25	8.25	7.75	8.75
Type	*A C*	*A C*	*A C*	*A C*	*A C*	*B*	*B*	*B*	*B*	*B*

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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PAPERS.

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THE CONSTRUCTION OF THE LORAIN DRY DOCK AND SHIPYARD OF THE CLEVELAND SHIP-BUILDING COMPANY.

By JAMES RITCHIE, M. Am. Soc. C. E.

TO BE PRESENTED APRIL 20TH, 1898.

In November, 1896, the Cleveland Ship-Building Company, of Cleveland, O., decided to remove its shipyard from its location in Cleveland, on account of the necessity of enlarging the plant and of constructing a dry dock in connection with the same, in order that repairs to vessels could be made within the limits of its own territory. With this object in view, the author was directed to make certain preliminary investigations of property in Loraine, O., located at the mouth of Black River, 26 miles west of Cleveland, to determine its availability for such purposes. These investigations were commenced in December, 1896, and consisted of a series of soundings and borings on the property to determine the character of the underlying strata. The soundings were made with a 3-in. test augur and extended to a depth of 51 ft. in the deepest place. The tests were made as nearly as possible on the center line of the proposed dry dock, on the easterly side of Black River and about half a mile from the lake.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

The surface of the ground consisted of marsh mud, the level of the highest point of the marsh being about 1 ft. above the ordinary stage of water in the river, which is the zero of the Government gauge. The marsh was covered with a heavy growth of reeds and grass, and was highest close to the river, along the bank of which it was possible to walk at all times, while 100 ft. back from the river the marsh was lower than the water level and was soft and miry. The depth of the marsh mud varied from 7 to 15 ft., as shown upon the profile of the soundings (Fig. 1). Below the mud was an inferior sort of bluish clay, which increased in stiffness with every foot of depth. At several of the test holes a small stratum of gravel was found, but it was in all cases below the level of the bottom of the proposed dry dock and was not deemed dangerous to the work. The result of the investigations was such that the location was fixed and the plans and specifications prepared by the author during the months of January and February, 1897.

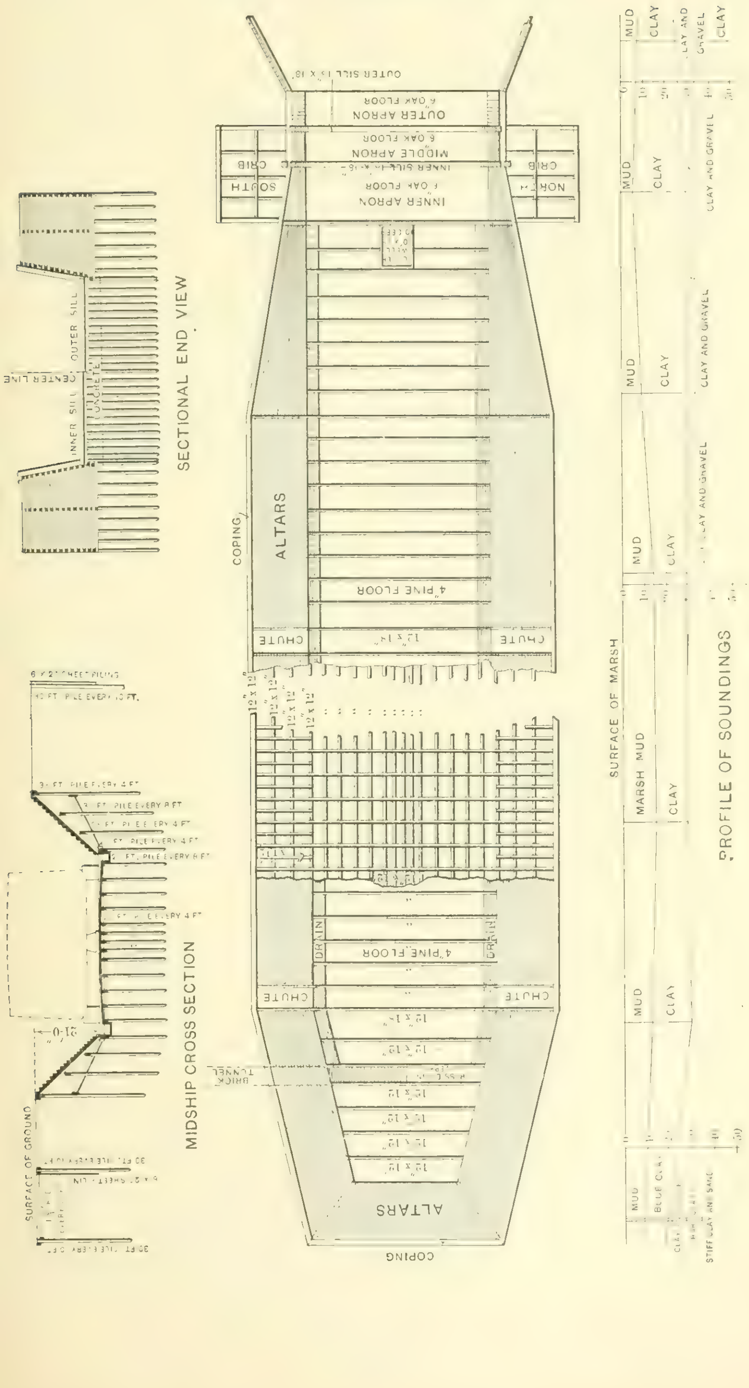
The first work to be commenced was the dry dock, and it will be described to completion, without reference to the remainder of the work, which was going on at the same time, and which will be described in the latter portion of this paper.

The plans prepared contemplated a dry dock 560 ft. in total length, 98 ft. wide at the top, between copings, 56 ft. wide at the bottom and 23 ft. total depth. The entrance was to be 60 ft. wide at the bottom and 66 ft. at the top, with a depth of 17 ft. of water on the sill at the ordinary stage. The plans were modified by decreasing the depth of the dock to 21 ft., which increased the bottom width to 59 ft. The width of the entrance was not changed. Fig. 1 shows the general plan and sections of the dock, the only change from the original being that the width on top, in the clear, between copings, is 96 ft.

Construction was commenced on February 22d, 1897. The first work consisted in enclosing three sides of the proposed dock by a water-tight protection of sheet piling, placed 25 ft. away from the top of the dock. This is shown in Fig. 1 of Plate VI. This protection was built of a single line of oak piles, 30 ft. long, driven 10 ft. apart and connected by a waling timber of 10-in. x 12-in. Norway pine, drift-bolted to each pile on the side away from the dock with 1-in. drift bolts. Outside of this waling piece was driven a row of sheet piling 6 ins. thick and from 20 to 26 ft. long, double grooved on the

Fig. 1.

Fig. 2.



SECTIONAL END VIEW
OUTER SILL
CENTER LINE
INNER SILL
CONCRETE
MIDSHIP CROSS SECTION
SURFACE OF GROUND
OUTER SILL 15" x 18"
OUTER APRON
6" OAK FLOOR
MIDDLE APRON
6" OAK FLOOR
INNER APRON
6" OAK FLOOR
SOUTH
CRIB
NORTH
CRIB
ALTARS
COPING
4" PINE FLOOR
CHUTE
12" x 14"
12" x 12"
DRAIN
BRICK
TUNNEL
ALTARS
COPING
MUD
CLAY
CLAY AND GRAVEL
MARSH MUD
SURFACE OF MARSH
BLUE CLAY
STIFF CLAY AND SAND
PROFILE OF SOUNDINGS

Fig. 1.

edges and tongued with 2-in. x 4-in. pine, spiked into one piece of the sheeting. The tongues were of well-seasoned lumber, and, when in place, it was found that they had so swelled as to make the joints thoroughly water tight. While this work was in progress, the upper row of piling for the dock itself was driven; the piles being of white oak, 35 ft. long, spaced 4 ft. apart, and extending along the north and south sides and the east end of the dock.

On the completion of this work the space between the piles last mentioned was dredged out to a depth of 24 ft., excepting that at the entrance, where it was proposed to place the coffer-dam, the dredging was only carried to a depth of 9 ft., or just sufficient to permit the entrance of the dredge and scows. The dredging left the banks of the excavation standing at a slope of about 1 to 1, and the top row of piling was thereby protected, and served also to some extent to hold the upper stratum of muck from sliding in.

The coffer-dam was constructed across the opening, extending from the north to the south line of the protection sheet piling. It was built by first driving two rows of 30-ft. oak piles 10 ft. apart; the piles being spaced at 10 ft. centers in each row, and connected by a 10-in. x 12-in. waling timber; and secondly, by driving a double wall of 6-in. tongued and grooved sheet piling, 26 ft. long, back of the waling timbers. The two walls were braced together by 8-in. x 8-in. diagonal braces and $1\frac{1}{2}$-in. iron rods, after the manner of the lateral bracing of a Howe truss bridge. On the river side of the dam, and 25 ft. from the same, was driven a row of 35-ft. oak piles, secured to each other by 12-in. x 12-in. timbers back of the piles. To these timbers the piling of the coffer-dam was anchored by $1\frac{1}{2}$-in. iron rods every 3 ft. The space between the anchor piles and the coffer-dam was filled with clay, which was banked up against the anchor piles to a height sufficient to protect the dam from the action of waves or from changes of level in the river. As the depth of water on each side of the dam was only 9 ft., the pressure upon the dam after pumping out was not excessive, as the natural ground below sustained the main weight. As soon as the pumping had progressed far enough, a second row of piles was driven on the side towards the dock, and the inside of the dam was shored up against these by timbers. The pumping was done slowly and was suspended at intervals to admit of driving the intermediate rows of slope piling, which was done from a floating driver. As the pumping

progressed and the water receded from the sheet piling, anchor piles were driven, and the sheet piling was secured thereto by 1¾-in. rods. The falling water caused the banks to cave off to some extent, and in two places the upper slope rows were disturbed and had to be re-driven for about 150 ft. on the south, and 200 ft. on the north side. There was practically no leakage through the sheet piling, until after the caving above mentioned, but water came in through the bottom of the dock, accompanied by marsh gas in considerable quantities. The water appeared to come up from a deep stratum and was 27° colder than the water in the river.

A second set of soundings was made in the bottom of the dock, and a vein of gravel was found through which the water appeared to come. A second row of sheet piling was driven inside the first row and to a depth of 20 ft. below the bottom of the dock, and this appeared to cut off a portion of the inflow. The remainder, being not enough to impede the work of construction, was left to be taken care of as needed.

The removal of the loose material left after dredging, and which had slid in during the pumping, and the work of bringing the bottom to the required grade, was commenced at the entrance of the dock in order that that portion might be first protected. The excavation was then continued toward the head of the dock at as nearly as possible the same rate as the pile-driving could be done. The material excavated was lifted out by derricks, loaded into mud scows and dumped in the lake. A portion of the excavated material was dumped by the derricks upon the adjoining marsh north of the dock. This was a mistake, as the weight of the material caused a break in the sheet piling at the same point as (previously stated) the caving off occurred, and the bottom of the dock appeared to be heaved up and to slide over the center from the north to the south side. At the same time, the surface of the marsh above sunk about 3 ft. This difficulty was finally surmounted, although it caused considerable trouble by increasing the amount of excavation, but as soon as the piling of the bottom was driven and the transverse timbers were put in place, this upward movement ceased.

The pile-driving of the bottom was commenced at the entrance of the dock, under the abutment cribs, and was continued toward the head of the dock as rapidly as the excavation allowed. There were two drivers in the dock, and each was employed constantly. A 2 000-lb.

hammer was used in driving the piles, and while at first they drove very easily, they finally gave indications of good bearing. Where it was thought best, longer piles than those called for by the plans were used. Extra piles were driven around all places where water appeared in the bottom, and the result was that nearly all the springs were stopped by the consolidation of the ground, only three or four leaks being left to the last, and these were led into the permanent drains and taken off by the pumps.

The piles under the slope timbers are of white oak and are driven in three rows on each slope, being 35, 30 and 25 ft. long, respectively. Those under the bottom of the dock and under the cribs at the entrance are 20 ft. long, and those under the aprons 25 ft. long. These are of various woods, such as beech, elm, pin-oak, hard maple, etc. The specifications required all piles to have a diameter of 12 ins. at a distance of 3 ft. from one end, and a diameter of not less than 6 ins. at the other end, in a length of 35 ft., and proportionally for shorter lengths.

The piling of the bottom consists of fifteen longitudinal rows, the five rows under the center and one under the foot of each slope having the piles spaced 4 ft. apart, and the other eight rows having them 8 ft. apart. Each row is capped with 12-in. x 12-in. oak, the timbers in adjoining rows breaking joint with each other. Upon these longitudinal timbers are placed the transverse timbers which support the keel and bilge blocks. These are 12 ins. x 18 ins. and 64 ft. long, placed every 8 ft. and notched 2 ins. over the longitudinal timbers, so as to brace the rows of piling. These timbers are of Douglass fir, from the Pacific coast. They extend clear across the bottom of the dock and support the main slope timbers. Between each transverse timber at the foot of the slopes there is placed a 14-in. x 14-in. timber framed into the 12-in. x 18-in. timbers and resting on top of the 12-in. x 12-in. longitudinals. This is to receive the intermediate slope timbers, and at the same time close the foot of the slopes against the earth behind the same.

The three rows of slope piles are also capped longitudinally with 12-in. x 12-in. oak, and each row is independently braced to the transverse timbers of the dock, so that any inward movement of either will be resisted by the bottom of the dock and the opposite slope. These are additional to the regular slope timbers which also brace the piling.

The slope timbers are 12-in. x 12-in. oak at every transverse bottom timber, and 8-in. x 12-in. oak at intermediate points. These timbers are framed into their supports, and form an additional brace to the top as well as serving their primary purpose of supporting the altars.

The slope timbers are covered with 1-in. rough pine boards nailed to each timber. At intervals of about 100 ft., along each side of the dock, chutes are constructed, by placing two 8-in. x 12-in. oak timbers on top of two of the 12-in. x 12-in. slope timbers, and filling in between them with 6-in. oak plank. These are to be used for putting material for repairs in and out of the dock without injury to the altars.

The entrance to the dock is protected by two abutment cribs, each 32 ft. x 28 ft. and 24 ft. high, resting upon 48 piles. The capping of the piles and the two upper courses of timber in the cribs are of white oak; the remainder of the timber is hemlock. The outer walls are of 12-in. x 14-in., and the inner walls of 12-in. x 12-in. timbers. The framing is dovetail work. All the courses are drift-bolted with 1-in. by 30-in. drift bolts, and all the joints are caulked with pitch and oakum. The sills and jambs for holding the gate in place are 18-in. x 18-in. oak, well braced and bolted in position. The jambs are sunk 2 ins. into the walls of the cribs, and are bolted through the walls and through 12-in. x 18-in. inside pieces of oak. The apron timbers, of 12-in. x 12-in. oak, run across the entrance, and are built into the cribs at each end. They rest upon piles between the cribs. There are two rows of piles and timbers under each sill, and these are braced together and to the adjoining timbers with 12-in. x 12-in. oak braces. Outside of the outer apron there is a line of 6-in. tongued and grooved sheet piling, 20 ft. long, extending across the entire opening and returning at each end to the face of the cribs, where it is connected to the line of sheet piling, 6 ins. thick and 30 ft. long, which is driven from the north end of the north crib to the south end of the south crib, and thence carried to the protection sheet piling of the dock. A third row of sheet piling, 30 ft. long, extends from crib to crib in front of the inner sill, thence coming forward along the cribs to the sheet piling in front of them.

The entire apron, inside of the outer row of sheet piling and to the inside line of the cribs, is filled in with cement concrete, made of broken limestone and American Portland cement. The depth of this concrete varies from 4 ft. at the outer apron and 5 ft.

under the middle apron to about 2 ft. at the inside of the inner apron. The cribs are filled with puddled clay. The space behind the slope timbers of the dock, and the entire bottom of the dock between the piling and the timbers, is filled with well puddled clay. As soon as the clay was filled in behind the slopes and in the bottom, the floor of the dock and the altars on the slopes were put in. The dock floor consists of 4-in. pine plank laid on furring strips on the longitudinal timbers, so that the floor is from 4 to 6 ins. lower than the top of the transverse timbers. The floor pitches from the center to each side, and the drains, built of 2-in. plank, run along each side, and empty into a cross drain at the east end of the dock. The cross drain is covered for half its length, and the remainder is protected by a grating of 1-in. iron bars, having 2-in. openings between them. This drain empties into a brick tunnel, 5 ft. in diameter and 96 ft. long, leading to the pumps.

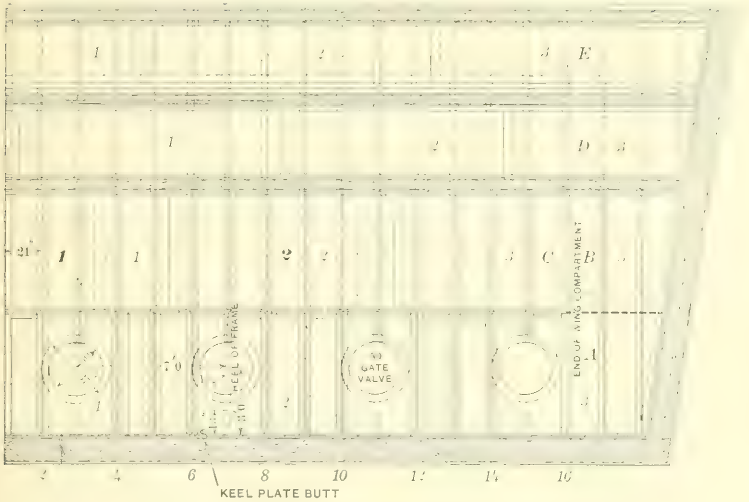
The altars or steps of the slopes are made by cutting diagonally through 10-in. x 14-in. timbers, so as to leave two edges, making a 10-in. rise and a 10-in. tread, and a third edge 2 ins. wide, to rest upon the step below. These altars are of white pine, and are spiked to the oak timbers with 12-in. spikes.

All the oak timber is drift-bolted to the piling, and to other timbers with 1-in. drift bolts of such lengths as the sizes of the timbers required.

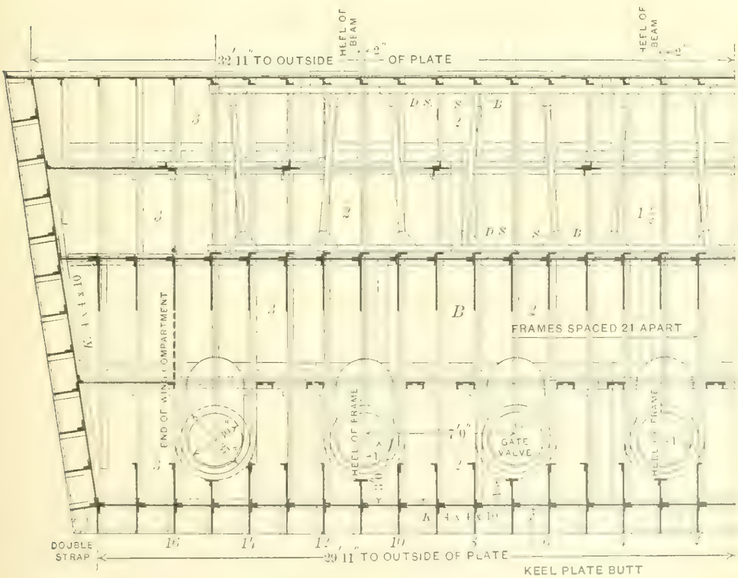
The coping of the dock consists of two pieces of 12-in. x 12-in. oak, laid side by side, and resting partly upon the upper altar and partly upon the cap of the upper slope row of piles, being drift-bolted to the cap and edge and bolted to each other.

The rudder well is located just inside of the inner apron and is 10 ft. wide, 15 ft. long and 10 ft. deep. It is protected by 6-in. tongued and grooved sheet piling, 20 ft. long, securely bolted to the timbers of the dock. It has a concrete bottom 2 ft. thick.

The gate is of steel, and is shown in Figs. 2 and 3. It is 62 ft. long on the bottom, 68 ft. on the top, including the jambs, and is 19 ft. high. The keel and jambs are made of two 15-in. channels, braced with plates and angles, and protected on the outside with 6-in. x 14-in. oak timbers, to which are fastened rubber gaskets. The gate is fitted with valves for filling and emptying the various compartments, and also with eight 30-in. gate valves for filling the dock.



HALF ELEVATION



HALF SECTION

FIG 2.

The pumps are located in the engine-room of the power house, directly over the well into which the tunnel empties. They are two in number, each having a 30-in. discharge, and a combined capacity of about 50 000 galls. per minute. Each pump is operated independently by a single engine. The well is 6 ft. x 10 ft. and 17 ft. deep. Its top is level with the engine-room floor, which is 10 ft. below the top of the dock, and is covered by a cast-iron plate, 10 ft. x 14 ft., bedded in cement, upon the 24-in. brick walls of the well, and bolted down to the same with 28 bolts 1 in. in diameter and from 5 to 8 ft. long. The suction pipes enter the well through packing boxes in this plate. There is also a manhole and a ladder for entering the well. The construction of the timber work was completed and the keel and bilge blocks placed in position on January 14th, 1898, but the filling in of clay behind the slopes was not finished then. The water was let into the dock through a sluiceway in the coffer-dam on the 15th and 16th of January, and on the 17th the work of removing the coffer-dam was commenced. This was completed on January 23d, and on the following day the gate was put in place and the pumps started. The dock was pumped out in two hours; and it was found that the gate and cribs and all the work were practically water tight. On January 26th the water was let in through six of the eight valves, and the dock filled in thirty-five minutes. On the same day the steamer *Sir William Fairbairn*, of the Bessemer Steamship Company, was successfully docked in two hours, and without injury to either the vessel or the dock. See Fig. 2, Plate VI.

The work of filling in behind the slopes is still going on, but, on account of the weather, may not be completed for some time. The material used for filling and puddling was yellow clay obtained from excavations in the bluffs back of the marsh. The excavated material was also used in bringing the marsh to grade for the construction of the shipyard.

The power house, as shown in Fig. 4, includes the machine shop, 40 ft. x 30 ft.; the boiler room, 32 ft. x 46 ft.; the coal room, 10 ft. x 46 ft., and the engine room, 36 ft. x 56 ft. There is an iron stack 6 ft. 8 ins. in diameter, and about 75 ft. high. The power house is of brick with concrete foundations. Considerable difficulty was experienced in its construction on account of its proximity to the dry dock and the desire of the company to commence work in the ship

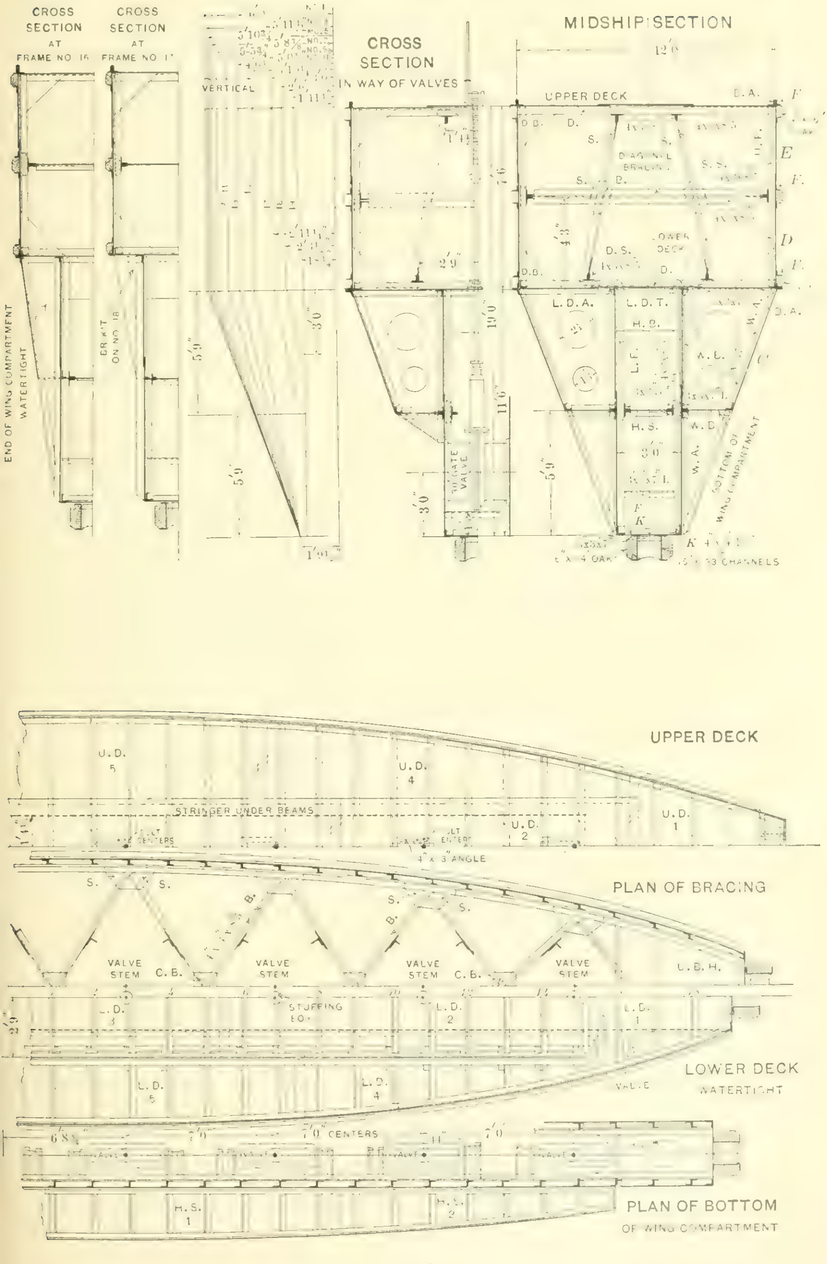

CROSS SECTION AT FRAME NO 16
CROSS SECTION
VERTICAL
CROSS SECTION IN WAY OF VALVES
MIDSHIP SECTION
UPPER DECK
END OF WING COMPARTMENT WATERTIGHT
L. D. A.
L. D. T.
H. S.
W. A.
UPPER DECK
U. D. 5
U. D. 4
U. D. 2
U. D. 1
STRINGER UNDER BEAMS
4" x 3" ANGLE
PLAN OF BRACING
VALVE STEM
C. B.
STUFFING
L. D. 3
L. D. 2
L. D. 1
LOWER DECK
L. D. 5
L. D. 4
CENTERS
H. S. 1
PLAN OF BOTTOM

FIG. 3.

yard by October 1st, 1897. This compelled the building of the power house before the walls of the dock could be put in, and trouble occurred when the bottom of the dock was suddenly raised, as described previously. This caused a movement in the foundation of the boiler room, but it was not sufficient to render it dangerous to the building. It was first proposed to construct the engine-room just west of the

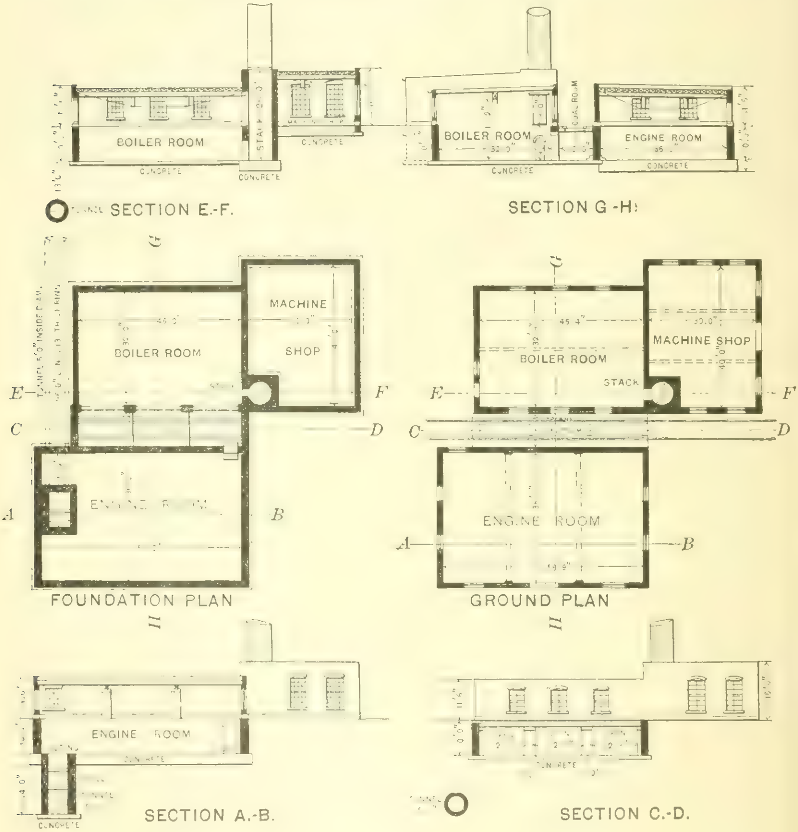


FIG. 4.

boiler room and close to the dock, and the excavation was made for the pump well with this in view; but in making this excavation a sudden flow of gas and water drove the men out, and it was decided to change the location to that shown on the plan. In the work thereafter there was no difficulty. The engine-room contains, besides the pumps previously described, two other engines, one being an air com-

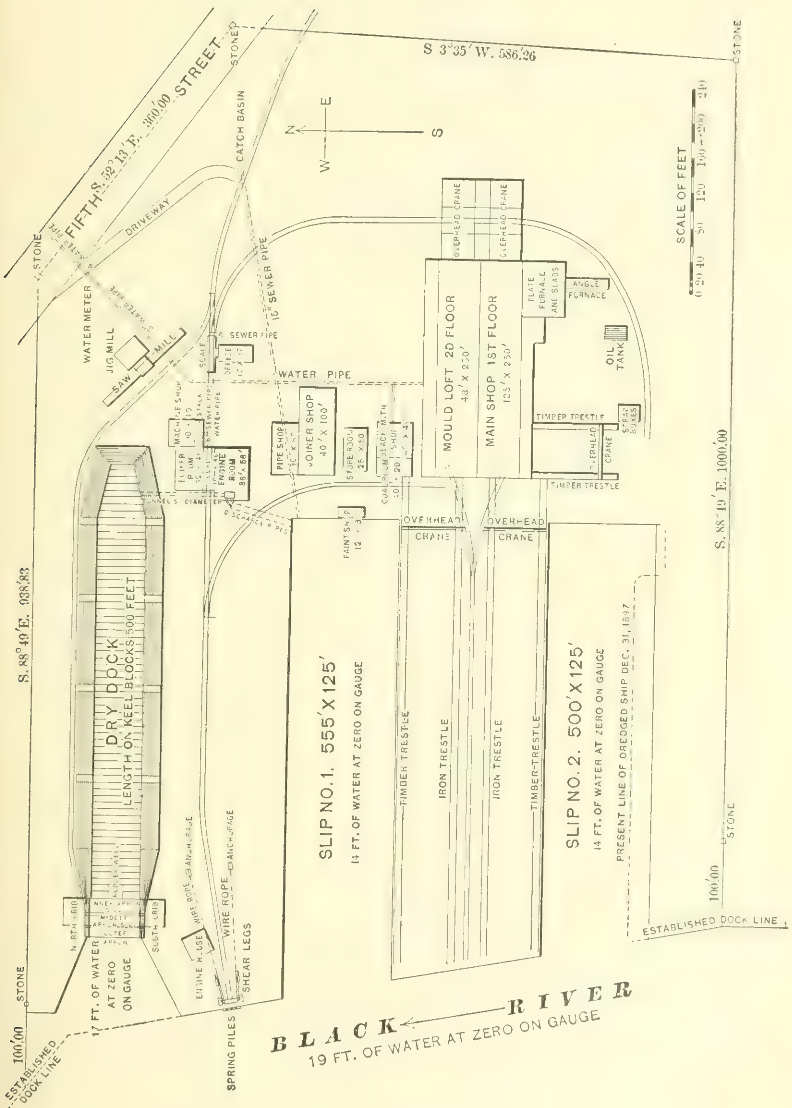

S 3°35′ W. 586.26
STONE
FIFTH S. 52°13′ E. 360.00 STREET
CATCH BASIN
DRIVEWAY
WATER METER
JIG MILL
SAW MILL
15" SEWER PIPE
SEWER PIPE
WATER PIPE
SCALE
MACHINE SHOP
ENGINE ROOM 36′ x 88′
PIPE SHOP
JOINER SHOP 40′ x 100′
STORE ROOM
PAINT SHOP
OVERHEAD CRANE
MOULD LOFT 2D FLOOR 43′ x 250′
MAIN SHOP 1ST FLOOR 125′ x 250′
PLATE FURNACE AND SLABS
ANGLE FURNACE
OIL TANK
TIMBER TRESTLE
SCRAP BOXES
OVERHEAD CRANE
SCALE OF FEET
S. 88°49′ E. 1000.00
S. 88°49′ E. 938.83
DRY DOCK
LENGTH ON KEEL BLOCKS 500 FEET
SLIP NO. 1. 555′ X 125′
14 FT. OF WATER AT ZERO ON GAUGE
TIMBER TRESTLE
IRON TRESTLE
IRON TRESTLE
TIMBER-TRESTLE
SLIP NO. 2. 500′ X 125′
14 FT. OF WATER AT ZERO ON GAUGE
PRESENT LINE OF DREDGED SHIP DEC. 31, 1897
ESTABLISHED DOCK LINE
100.00
NORTH CRIB
SOUTH CRIB
17 FT. OF WATER AT ZERO ON GAUGE
ENGINE HOUSE
WIRE ROPE
SHEAR LEGS
SPRING PILES
BLACK RIVER
19 FT. OF WATER AT ZERO ON GAUGE
ESTABLISHED DOCK LINE

FIG. 5.

pressor and the other an electric generator. The compressor furnishes air at 80 lbs. pressure to the forges, riveters and pneumatic hoists in the shops. The generator furnishes electricity for operating the rolls, punches, shears, drills, over-head cranes, wood-working machinery, and the lights in the yard and shops.

The boiler room contains three boilers, with space for a fourth should it be needed.

The coal room has a capacity of about 100 tons, and is arranged so that cars can be run overhead and unloaded through the bottom.

The machine shop is for light repairs only, the company having its large machine shop at Cleveland.

The main shops of the shipyard are in the steel building shown in Fig. 5 at the end of the space between the slips. This building is 125 ft. wide and 250 ft. long, and has a second floor used for a mould loft, 48 ft. wide and 250 ft. long. There are nine bents 31 ft. 3 ins. apart, and the intervening spaces are filled with the tools and machinery for punching, shearing and shaping the steel. Two over-head cranes travel the length of the building, and at the east end run out upon a trestle work over the track upon which material is received. Attached to the columns of the building are jib cranes furnished with pneumatic hoists which handle the material at the various tools. At the south-east corner of the building there is an extension to the south, containing the heating furnaces, operated by crude oil, and the slabs for bending plates, angles, channels, etc. At the southwest corner and outside of the building there is an over-head crane to handle the frames and to tend the pneumatic riveter.

The mould loft is provided with wood-working machinery, and here are prepared the patterns for the different parts of the ship. The floor is a drawing board upon which the lines of the ship are laid down.

The blacksmith shop, 40 ft. x 40 ft.; the coal room, 20 ft. x 40 ft.; the joiner shop, 40 ft. x 100 ft., and the pipe shop, 30 ft. x 60 ft., are supplied with the necessary tools and machinery for their work, but offer no special features of interest. The same is true of the saw mill, which has a gang saw and a jig saw for timber work on wooden ships.

All the machinery not previously mentioned as run by compressed air is operated by electricity, except the hammer in the blacksmith shop, which is run by steam.

The office, 37 ft. x 17 ft., contains rooms for the superintendent and his clerks, a weighing room and a toilet room on the lower floor, and a parlor, dining-room and kitchen up stairs.

The shears, for lifting engines, boilers and other heavy articles into and out of vessels, are located on the river front between the dry dock and Slip No. 1. Their capacity is 60 tons.

The above-mentioned structures are all shown on the general plan (Fig. 5). There are two slips which are dredged out to a depth of 14 ft. The sides and ends are protected by piling and sheet piling anchored back to another row of piles as shown in Fig. 1.

Between the two slips are two berths for building ships. There is also room for another berth between "Slip No. 1" and the dry dock, and also between "Slip No. 2" and the south boundary of the property, but at present only two are equipped for use. The ships, when under construction, are supported upon blocking resting upon 12-in. x 12-in. oak timbers, which, in their turn, are supported by and bolted to piling. There are six rows of timbers under each ship. Over each berth there is a traveling crane operated by electric motors, and supported on the iron and timber trestles indicated in Fig. 5. The tops of these trestles are 35 ft. above the ground. The cranes handle the material from the shop or from tramcars on the railroad track below, and place the plates or frames where desired on the ship. The iron trestle is fixed in position, and the timber trestle is movable. When a ship is ready for launching, the timber trestle is removed by the overhead crane and carried back out of the way, leaving the side of the ship free towards the slip. Vessels built upon the lakes are launched sideways.

There are at present under construction two steel vessels, one being 450 ft. long with 50 ft. beam, and the other 438 ft. long with 48 ft. beam.

The result of the work has been very satisfactory, so far as it has at present been tested by actual use, and the indications point to its becoming more satisfactory as its various parts become settled into place and the operatives become familiar with them.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

FLUSHING IN PIPE SEWERS.

By H. N. Ogden, Jun. Am. Soc. C. E.

To be Presented May 4th, 1898.

The use of flush-tanks in connection with small pipe sewers, which has been made an integral part of the "Separate System" and generally adopted in systems caring only for house sewage, is attended with much uncertainty. In such systems it is generally specified that a flush-tank be placed at the head of every lateral, each tank being so regulated as to discharge at least once in 24 hours. The relation between the size of the sewer pipe and the amount of water used in a flush is not given, nor is the influence of grade discussed. The general law is laid down that all laterals, regardless of size, grade, or contributing population, must be supplied with flush-tanks in order to secure a self-cleansing flow in the laterals and to maintain the integrity of the system.

The financial burden of such a requirement is evident. As an example, it may be cited that in the plans for the sewerage system of Ithaca, N. Y., in which this requirement of flush-tanks was thoroughly complied with, even for the 12% grades, no less than 131 flush-tanks were required in 25.3 miles of sewers, or one for every 1 020 ft. The relative importance of the flush-tanks may also be seen by comparing

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

the actual cost of the sewers with the estimated cost of the tanks. The cost of the sewers, viz., the sum of the amounts of the several contracts was $81 000, and, estimated at $50.00 each, the flush-tanks would cost $6 550, or more than 8% of the cost of the system. It would seem, then, that the cost of flush-tanks is by no means insignificant, but that their use increases the cost of the separate system by nearly one-tenth, besides introducing a permanent charge, both for water used and for intelligent care in maintenance. That these annual charges are no bagatelle will be apparent by again referring to the case of Ithaca. Assuming that the tanks required are of only 150 galls. capacity, a minimum amount, discharging but once a day, the water required is 19 650 galls. a day. Twenty cents per 1 000 galls. (the amount charged in Ithaca*) is a fair average amount, and at that price the daily charge for water is $3.93 or $1 434.45 per year. Adding to this $600 per year as the wages of a mechanic, whose constant attention is found by experience to be necessary in examining and readjusting the tanks, the total annual charge is $2 034.45. This, capitalized at 6%, gives $33 908, and, added to the $6 550, gives $40 458 as the total expenditure on account of flush-tanks in a sewer system costing for pipe laid $81 000. Surely the item of flush-tanks is an important one, and should be carefully examined, so that if the conditions of the sewer grade, for example, modify the necessity for tanks, or if the amount of water is a function of the time interval between flushes, or of the size of the pipe, it may be known in order that the large proportionate cost of flushing may be reduced to what has been found by careful investigation to be an absolute minimum.

That the requirement given above is felt by present-day engineers to be largely in excess of necessity is sufficiently evident from a study of the paper by F. S. Odell, M. Am. Soc. C. E., entitled "The Separate Sewer System Without Automatic Flush-Tanks,"† and the subsequent discussion, in which the author says that at Mt. Vernon, N. Y., no flush-tanks are used, and that, while hand-flushing by means of fire hose is practiced at intervals of six months, even this infrequent flushing does not appear necessary, as examination of the sewers invariably shows a very wholesome and satisfactory condition. In the discussion very little positive evidence is given, but the experiences recorded go chiefly to show that while automatic flush-tanks do not in them-

* *Manual of American Water Works*, 1897.

† *Transactions* Am. Soc. C. E., Vol. xxxiv, page 223.

selves make the separate system practicable, there is, nevertheless, a need, under certain conditions, for flushing, those conditions being as yet not fully determined.

The questions the answers to which are essential for an intelligent disposal of flush-tanks on a sewer system are four, viz.:

1. What is the relation, if any, between the grade of the sewer and the necessity for automatic flush-tanks?

2. Assuming a need for automatic tanks, how does the grade of the sewer affect the amount of water required, and what is the proper amount to be used?

3. How often should tanks be discharged?

4. What effect does the substitution of a 6-in. for an 8-in. lateral have on the necessity for tanks and on the amount of water to be used?

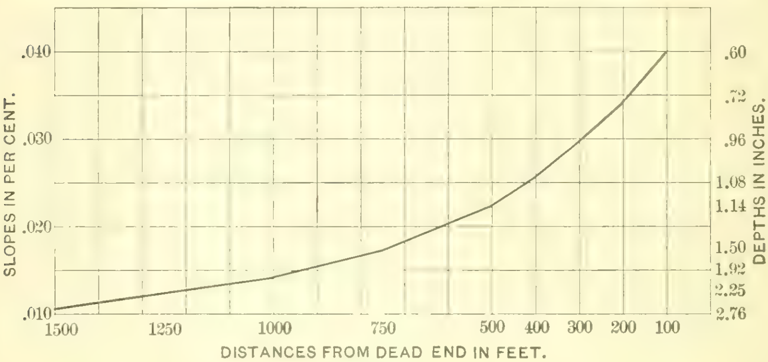


FIG. 1.

Before attempting to answer these questions, it will be well to look at the subject broadly, and consider the hydraulic problem involved. Sewage is water carrying in suspension less than 1 part in 1 000 of solid matter, and sewers are supposed to be so laid that the resulting velocity of flow is sufficient to keep this solid matter in suspension. This suspending and scouring power probably depends on the velocity, and on the depth, of the sewage stream, and if either gets below a certain point, sedimentation will follow and a deposit take place. It is generally stated that a velocity of about $2\frac{1}{2}$ ft. per second is required; but the effect of depth is neglected. At the lower end of a 6-in. lateral the depth and velocity are assumed to be sufficient to prevent this sedimentation, but as the contributing population grows less toward the upper end, the depth and velocity decrease and

the transporting power of the stream falls so low as to allow the solid matter, brought into the sewer by the house drains, to become stranded. This deposit increases by gradual accumulation until the sewer is blocked, until the head from the backed-up sewage is sufficient to carry away the obstruction, or until the discharge of the flush tank (and here is seen its true function) takes up the obstruction and carries it to a point where the depth and velocity of the sewage will hold it in suspension. Table No. 1, and the diagram Fig. 1, are given to show the requirements in grade to maintain a velocity of $2\frac{1}{2}$ ft. per second in a 6-in. lateral, assuming a constant contributing population of 76 persons per 100 ft. of sewer, with a daily flow of 60 galls. per capita, and with the assumption of one-half flowing off in 6 hours.

TABLE No. 1.

Distance from dead end in feet.	Discharge in cu. ft. per sec.	Slope in ft. per foot.	Depth of flow in inches.
1 750	0.245	0.0103	3.00
1 500	0.210	0.0104	2.76
1 250	0.175	0.0123	2.25
1 000	0.140	0.0140	1.92
750	0.105	0.0174	1.50
500	0.070	0.0225	1.14
400	0.056	0.0256	1.08
300	0.042	0.0302	0.96
200	0.028	0.0342	0.72
100	0.014	0.0400	0.60

The diagram (Fig. 1) shows that, taking n equal to 0.013, and computing velocities by Kutter's formula, a grade of 1% is required for a 6-in. pipe half full for a velocity of 2.5 ft. per second, and that if the amount of flow constantly decreases, the depth of flow decreases also, and the grade, in order to maintain the same velocity, must be increased according to the diagram. The diagram is given for two reasons ; first, to show that by the accepted laws governing the transportation of material in flowing water, lateral sewers could be laid, theoretically, on such grades that no flushing would be necessary, since, with grades which continually increase toward the upper end, the corresponding velocities would always be equal to that required to transport matter in suspension ; second, that as the grade of the sewer increases, the distance from the upper end to the point where the stream reaches the velocity required to carry matter in suspension decreases, and so the aid required from flush-tanks is less. No value can be

placed on the grades given, as the diagram is based on the assumption of a house with five persons every 66 ft., and this is not always the case, but it is believed that there is a grade at or beyond which flush-tanks are not required, and that if the distance to which the flushing power extends is a function of the amount of water discharged, then this amount should be less on the steeper grades.

Referring again to Mr. Odell's paper, it is first noted that at Mt. Vernon, with grades of from 0.5% to 6% no flush-tanks are used, and a good hand-flushing twice a year answers every purpose.

In the discussion, Mr. Hering says that on light grades flushes of 200 to 300 galls. generally lose their flushing power after passing a few hundred feet through the pipe, and that sometimes after 500 ft. he had been unable to detect any difference in the flow due to the tank.

Mr. Kiersted writes that in one system designed by him he recommended flush-tanks only on laterals of less than 0.5% grade, and for five years the system has been in operation with but few stoppages.

Mr. Folwell writes that in his experience he has omitted flush-tanks on grades from 6% to 12%, and on the 6% grades no stoppages were discovered, nor were there any odors.

Mr. Le Conte intimates that flush-tanks as built do not answer their purpose, for where grades are light and the flush most needed, they do the poorest work; and the large quantity of water needed, to be effective, must be obtained by some other means.

Mr. Odell maintains that flushes of 200 galls. or less fail to flush a sewer properly, especially on flat grades where flushing is most needed.

A table by Mr. Allen shows that on grades greater than 0.5% a velocity of more than $2\frac{1}{2}$ ft. per second is maintained over 1 000 ft. from the flush-tank, but on lesser grades the velocity drops to 2 ft. or less within 600 ft.

In order to obtain an insight into general engineering feeling in the matter, and, at the same time, reap the benefit of any experience which was to be had, the author sent out on January 17th, 150 reply postals, reading as follows:

"Ithaca, N. Y., January 17th, 1898.

"Dear Sir:

"To aid me in deciding as to the necessity for flush-tanks for our sewer system, will you kindly answer the following:

"I.—Do you find flush-tanks a necessity, or is periodic hand flushing sufficient to keep sewers clean ?

"II.—Does the element of grade affect the question, and within what limits of grade are tanks required ?

"III.—Does your experience show any relation between the minimum amount of water required for effective flushing and the grade of the sewer ?

"Thanking you in advance for your kind assistance in this matter,

"I am, yours very truly,

"H. N. OGDEN,

"*Engineer, Ithaca Sewer Commission.*"

These postals were sent to those cities of between 10 000 and 60 000 population, in the New England and Middle Atlantic States especially, which were reported in *The Manual of American Water Works for* 1897 as having separate or sanitary sewers. Eighty answers were received, and the courtesy and good-will expressed in all was unmistakable and much appreciated. The same story was told by them in nearly all cases. "I would be pleased to answer your questions fully, but this is the best that I can do for you," or "This is only my idea, while I can readily understand that what you want is the result of actual experience," or "I cannot give you the desired information, but would be thankful to you if you would let me know the result of your inquiry." The results given below in a brief summary chiefly show how uncertain and vague is the knowledge on the subject, and how necessary some experiments and investigations.

In answer to question No. 1, whether flush-tanks were necessary, of the eighty replies seventeen had no opinion on the subject, and twelve had experience only with combined systems, but had, according to their replies, found no trouble in keeping the ends of their 10-in. and 12-in. laterals clean with rain or with hand-flushing. Twenty-six of the eighty used periodic hand-flushing and found it to answer every purpose, keeping the sewers clean and free from obstructions. Twenty-five either used flush-tanks or considered them a necessity for small pipe sewers. It was not possible in these last answers to separate actual experience from personal conjecture on the question, so that this number may include many hearsay opinions.

The evidence is not very clear. The fact that twenty-six used hand-flushing satisfactorily indicates that such flushing is sufficient. That it must be properly and regularly done, however, is made plain

by the fact that, out of the twenty-five believing in flush-tanks, nine had tried periodic hand-flushing, found it uncertain and irregular, and had put in flush-tanks, to secure proper attention. On the other hand, of the twenty-six believing in hand-flushing, two came to that opinion after becoming disgusted with the uncertainty of tanks.

To the second question, only twenty-three of the eighty ventured an opinion. Of these, eight thought that the grade did not affect the question, but that flush-tanks were as necessary on steep as on flat grades. One engineer explained his position by saying that while the velocity on the steep grades might be greater, yet as the depth would be less, the transporting power would be less, and therefore tanks were equally necessary. Of the fifteen who thought that tanks are not needed above a certain grade, six merely ventured it as an opinion, and nine fixed the limit at from 0.5% to 3 per cent. Four give 1% as the limit; one, 3%, and the other four give less than 1 per cent.

Only six replies were given to the last question, whether the amount of water in the flush-tank should be varied with the grade of the sewer. Of these, two thought that no difference should be made; three thought that less water could be used on the steeper grades, but had no definite opinion as to the relative amounts; while one well-known engineer, who has thoroughly studied the workings of the sewer system under his care, writes that he finds one flush daily on a 2% grade as effective as two flushes daily on a 0.5% grade, each flush of 300 galls.

The general conclusion from the replies is that occasional flushing on low grades, probably below 1%, is needed at the upper ends of laterals; that this may be accomplished, either by hand-flushing or by the use of automatic tanks; that if tanks are used, less care and vigilance are required in inspection and oversight, but if they are used, the periodic examination of the system, which should not be omitted, is apt to be irregular, and if a tank fails to work or if an obstruction occurs below the effect of the flush, a serious nuisance may result; that if hand-flushing is employed, a constant and regular inspection must be practiced, although actual flushing may be required only once a month or less. The amount of water needed in flush-tanks is not known, nor the relation between amount and grade.

With a view of obtaining more information on this apparently unstudied subject, the author carried on some experiments in the spring

of 1897. He was assisted by Mr. I. W. McConnell, C. E., who had been the writer's valued assistant on the construction of the Ithaca sewer system for two summer vacations. The results of the experiments have been recorded by Mr. McConnell in a thesis for the degree of Civil Engineer in Cornell University.

The sewers on which the experiments were made, and which were chosen so as to afford a variety of grade, with as long lines as possible, were all 8-in. pipe, and each had at the upper end a manhole about 4 ft. in diameter at the bottom. Flush-tanks of usual commercial size discharge at a rate of about 1 cu. ft. per second, and, by repeated experiment, the opening from the manhole into the sewer was reduced to such a size (about 5 ins.) that the rate of discharge varied from 0.89 cu. ft. per second for 4 ft. head in the manhole to 1.1 cu. ft.

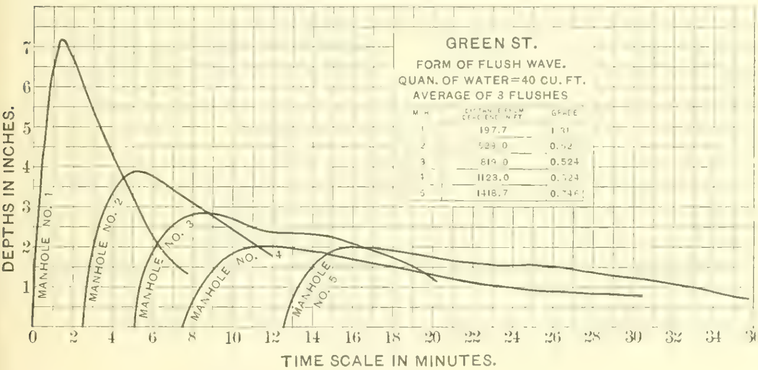


Fig. 2.

per second for 6 ft. head. These conditions it was thought approximated closely enough to the workings of a flush-tank. A 5-in. opening was cut in a pine board firmly held against the end of the 8-in. pipe; then a flat cover, 6 ins. in diameter and faced with rubber, was provided, which, placed over the opening and held there by a light stick braced against the back of the manhole, made an effective plug. The manhole was filled to any desired depth by means of fire-hose attached to neighboring hydrants, and then, by means of a cord fastened to the stick and to the cover, the contents of the manhole were discharged into the sewer. The capacity of the manholes at depths varying by 6 ins. was determined by measurement, so that by filling to the proper depth any desired amount of water could be discharged. The effect of the flush waves was then noted at the

successive manholes down the line. No determinations of the velocity of the wave were made, the effect being judged by the depth of the wave, and by the force shown in moving gravel, etc., placed in the different manholes. The wave depths were read by different observers stationed in the manholes, where they recorded as rapidly as possible (usually every seven seconds) the depth as marked on a thin vertical scale placed in the sewer. Figs. 2 to 5 show the wave forms and the progressive flattening as the wave gets farther and farther from the flush-tank.

To test the transporting power of the wave small brickbats and gravel of various sizes, coated with paint so as to be recognizable, were placed in the inverts at the manholes. A considerable growth of what was apparently of vegetable origin had become attached to the sides and bottom of the pipe, and the value of the flush in removing this growth was also noted. The order of procedure was to examine and note the condition of the line, and, after placing the gravel, etc., to make a number of flushes, each of 20 cu. ft., and note the results. Then, increasing the amount discharged to 30, 40, 50 and 60 cu. ft., the respective results were noted. Then, either the whole pipe was scraped by a rubber-edged piston-like cleaner, or merely the manhole inverts and about 6 ft. each way into the pipe, and the flushing repeated. The following tables give the results on the different lines:

TABLE No. 2.—Green Street Sewer.

Volume of flush.	Effects at				No. of flushes.
	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	
25 cu. ft...	Scoured clean.	Scoured clean.	No effect.	No effect.	1
30 " ...	"	"	"	"	1
40 " ...	"	"	Several stones started.	"	8
60 " ...	"	"	Small gravel generally started.	"	2
80 " ...	"	"	"	"	2
120 " ...	"	"	"	"	3

Before commencing the work, the examination of the pipe showed it to be practically clean, with no ground-water, except between the third and fourth manholes, where there was a stream perhaps ¼ in. deep. There were no house connections, but there was a small depth

of silt, and small pieces of cement left from construction, also a slight growth on the sides and bottom of the pipe. Gravel of all sizes placed in the pipe at the flush-tank was carried through to manhole No. 1 in two flushes of 25 cu. ft. each, the first flush alone not being sufficient. The gravel scoured out of the bottom of No. 1 manhole by the first flush was not brought to No. 2 until the 80-cu.-ft.

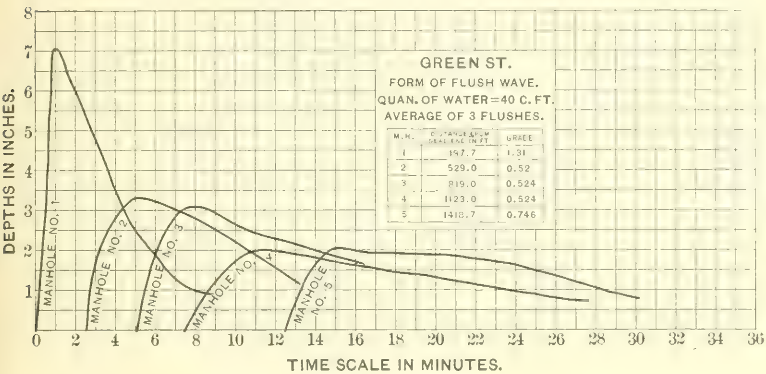


FIG. 3.

flush was put in, and no gravel scoured out of No. 2 was brought to No. 3 by any of the flushes. After the seventeenth flush as above, the pipe was thoroughly scraped and cleaned, and flushes eighteen to twenty-eight made. Similar results were obtained, except that the flushes carried the gravel about 200 ft. farther than before and seemed effective for that distance.

TABLE No. 3.—CAYUGA STREET SEWER.

Volume of flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of flushes.
30 cu. ft...	Scoured clean.	No effect.	No effect.	No effect.	3
40 " ...	"	Disturbed but not cleaned.	"	"	7
60 " ...	"	Partly scoured.	Some vegetable growth passed through.	"	2
80 " ...	"	Cleaned.	"	"	3

In Cayuga Street there were a few connections and little flow, so that the condition of the pipe was very foul; there was also a heavy vegetable growth in the pipes.

On Linn Street no comparative records could be made. The pipe was clean from the flush-tank to Manhole No. 1, and in this length there were no connections. From No. 1 to No. 2 it was slightly foul, and very foul the remainder of the length. There were two house connections on the line. Five flushes of 20 to 60 cu. ft. were made.

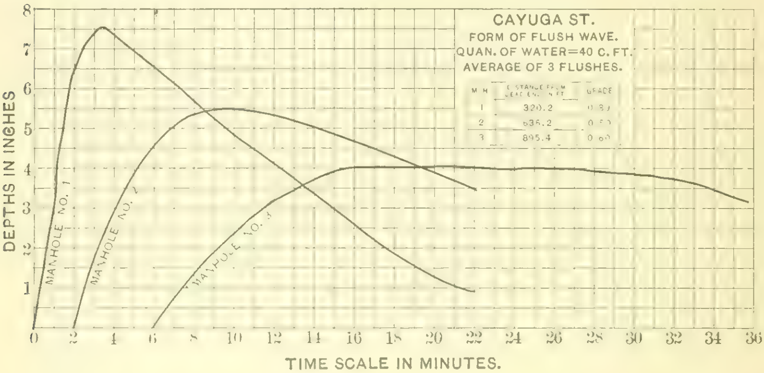


Fig. 4.

Each was very effective, one apparently as much so as another. All obstructions introduced were removed at once from manholes Nos. 1 and 2. A steady flow 1 in. deep from the hose carried everything forward at once to a point beyond No. 2 and to the flatter grade.

TABLE No. 4.—Aurora Street Sewer.

Volume of flush.	Manhole No. 1.	Manhole No. 2	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu. ft....	Cleaned.	Cleaned.	No effect.	No effect.	3
60 "	"	"	Disturbed.	Water dirty: some vegetable growth came through.....	7
80 "	"	"	"	A few stones disturbed.	2

TABLE No. 5.—First Street Sewer.

Volume of flush.	Manhole No. 1.	Manhole No. 2.	Manhole No. 3.	Manhole No. 4.	No. of Flushes.
40 cu. ft....	Cleaned.	No effect	No effect.	No effect.	5
60 "	"	"	"	"	3
80 "	"	"	"	"	2

On the Aurora Street line, the pipe was very foul, chiefly from a hospital connection at the upper end. The vegetable growth was large, and the accumulations of organic matter very evident.

On Buffalo Street, where the grade is about 12%, the effect of the flush was amazing. Where any sewage at all flows in the pipe, it is

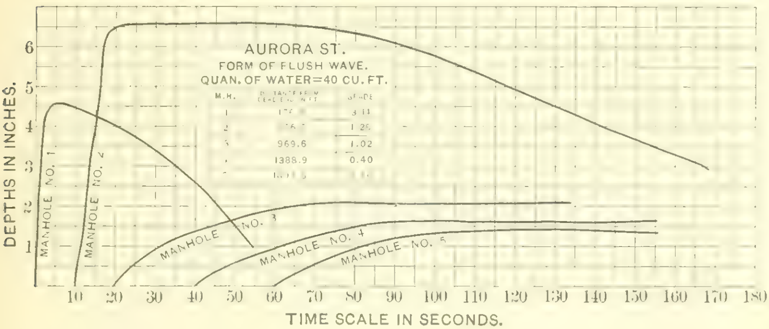


FIG. 5.

sufficient to remove all obstructions. A flush of any volume rushes down the hill at a high velocity, with piston-like action, and sweeps everything before it.

Table No. 6 gives the distances and grades between manholes on the lines used in the experiments.

TABLE No. 6.—DISTANCES AND SLOPES BETWEEN MANHOLES.

Description.	Green St.		Cayuga St.		Aurora St.		First St.		Linn St.	
	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.	Distance in feet.	Grade percentage.
Dead end to Manhole No. 1	298	1.31	320	0.89	177	3.14	371	1.00	331	2.94
Manhole No. 1 to Manhole No. 2	231	0.52	316	0.50	390	1.28	341	0.50	278	2.70
Manhole No. 2 to Manhole No. 3	290	0.52	259	0.60	413	1.02	394	0.57	317	0.50
Manhole No. 3 to Manhole No. 4	305	0.52	......	......	419	0.40	393	1.00	......	......
Manhole No. 4 to Manhole No. 5	296	0.75	......	......	417	0.80	......	......	......	......

The manager of the Van Vranken Flush-Tank Company gives his practice in proportioning the sizes of flush-tanks for any particular

sewer, as follows: The capacity of the reservoir should be equal to one-half that of a length of sewer in which the grade produces a rise equal to the diameter of the pipe; so that the Green Street line, 8 ins. diameter, and 0.5% grade, should have a discharge of half the volume of the pipe, $\frac{4}{3} \times 100$ in length, or 23 cu. ft.; and for a 1% grade one-half of that, or 11.5 cu. ft. He says further, and the statement has been confirmed by the author's work, that an 8-in. pipe on a 0.4% grade will flow about one-third full at a distance of 300 to 400 ft. from the tank discharging the above amount; and that on a 5% grade the water will come down as a solid piston for any discharge greater than 14 cu. ft.

The manager of the Pacific Flush-Tank Company writes that as a rule he does not interfere with engineers in their design for tanks, but, in his opinion, a flush of 175 galls. in a 1% grade is sufficient, and on any flatter grade twice that amount of water should be used, or, in other words, as he says, "long lines on flat grades require greater capacity of tanks than steep grades or short lines."

Conclusions.—The following conclusions are based upon data on this subject published previously; upon the experience of engineers in different parts of the country; upon the flushing diagrams published recently by J. W. Adams, and upon observation and the special experiments made in Ithaca; and it is believed that they are justifiable and are a safe guide as to the use of flush-tanks.

(1) Flushing of some sort is required at the upper ends of laterals, the frequency and amount depending on the number of house connections, on the carefulness or prodigality in the use of water by the householder, on the grade and size of the sewer, on the character of its construction, and on a mysterious something which defies definition, but which produces frequent accumulations in one line and does not affect another, apparently like the first.

(2) This variety in the conditions prevents any exact statement of a relation between the quantity of water which should be discharged from a flush-tank and the grade of a sewer, but it plainly indicates that the advantage of automatic flush-tanks lies in a general guarantee or insurance against accumulations in the upper part of the laterals, while periodic hand-flushing must be depended on only when in charge of a responsible, indefatigable and intelligent caretaker.

(3) Judging by the experience at Ithaca, and despite the statements of other engineers, it seems to the author that on grades of less than 1%

automatic flush-tanks are an economic necessity, even where water has to be paid for, the added expense of frequent hand-flushing more than off-setting the possible discharge of flush-tanks when not absolutely necessary.

(4) The volume of water discharged should not be less than 40 cu. ft., and the effect of the flush can hardly be expected to reach more than 600 or 800 ft. Below this point accumulations may occur which must be removed by hand-flushing and carried on to a point where the sewage flow has the necessary transporting power.

(5) On flat lines and where obstructions occur below the influence of the flush-tank, a second flush-tank, placed about 800 ft. from the first, will be more effective than increasing the first tank to a capacity of three times its original discharge.

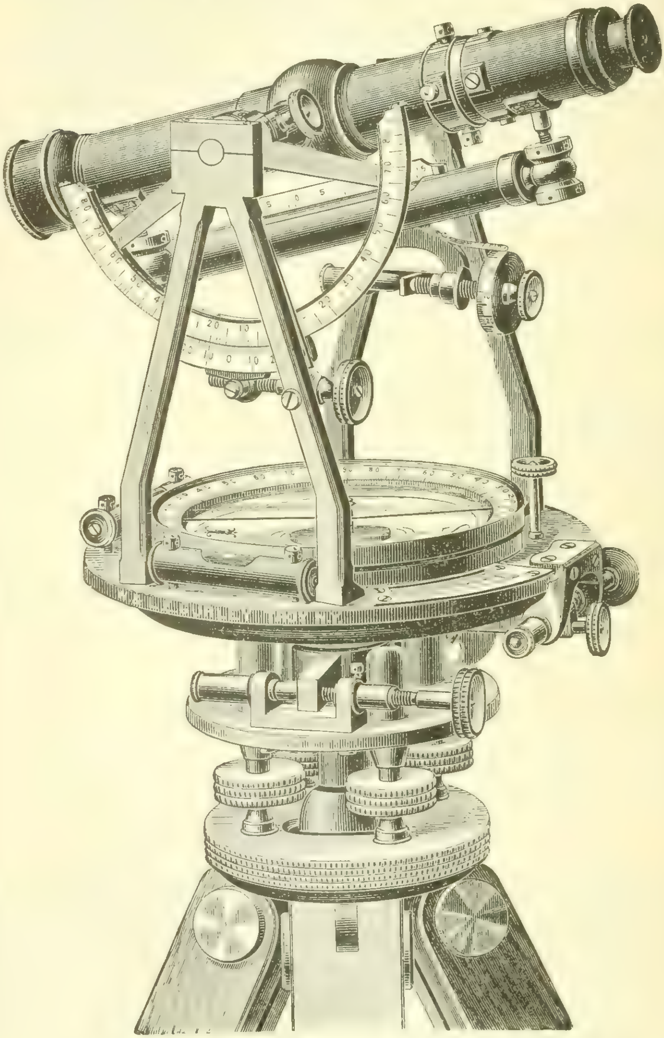
(6) The frequency of discharge should depend on the local conditions, but it is probable that the maximum interval depends on the practical working of the siphon, so that the usual prescription of once in twenty-four hours is a safe rule.

(7) If tanks are used on grades greater than 1%, 15 to 20 cu. ft. give as good results as larger amounts, with the same rule as to frequency of discharge.

(8) However, economy is best served, on grades above 1%, by omitting flush-tanks, and resorting to periodic hand-flushing at such intervals as experience shows to be necessary on the different lines. In most cases semi-annual or quarterly flushings, with a hose, are sufficient.

(9) On grades greater than 3% flush-tanks are unnecessary, and their installation is a waste of money.

(10) Hand-flushing should be performed and tanks discharged at night, as a flow of even an inch in a sewer offers a large resistance to the flushing action; while, with a pipe flowing half full, the effect of a flush-tank is scarcely visible.



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OF

CIVIL ENGINEERS

April, 1898

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PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publication.

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SOCIETY AFFAIRS.

PAPERS.

ILLUSTRATIONS.

The prices of publications are as follows: Proceedings, $6 per annum; Transactions, $10 per annum. Postage will be added when they are sent to foreign countries.

American Society of Civil Engineers.

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Term expires January, 1899:	*Term expires January, 1900:*
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JOHN F. WALLACE.	FREDERIC P. STEARNS.

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Treasurer, JOHN THOMSON.

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Term expires January, 1899:	*Term expires January,* 1900:	*Term expires January,* 1901:
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DANIEL BONTECOU.	HENRY S. HAINES.	JOHN KENNEDY.

Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open from 9 to 22 o'clock every day, except on Sundays, when the hours are from 14 to 19 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

April 6th, 1898.—The meeting was called to order at 20.20 o'clock, President Alphonse Fteley in the chair; Charles Warren Hunt, Secretary, and present, also, 100 members and 23 visitors.

The minutes of the meetings of March 2d and 16th, 1898, were approved as printed in *Proceedings* for March, 1898.

A paper by George Hill, Assoc. M. Am. Soc. C. E., entitled "Steel Concrete Construction," was presented by the author. Correspondence on the subject from Messrs. A. L. Johnson and J. B. Johnson was presented by the Secretary. The paper was discussed orally by Messrs. Mansfield Merriman, G. B. Waite, H. F. Dunham, R. W. Lesley, J. S. Merritt, F. S. Washburn, C. Tomkins, J. F. O'Rourke and George Hill.

Ballots were canvassed and the following candidates declared elected:

As Members.

William Wright Harts, High Bridge, Ky.
Harry Monmouth Herbert, Bound Brook, N. J.
Archibald Johnson, St. Paul, Minn.
Henry Francis Labelle, Montclair, N. J.
Henry Prentice Morrison, West New Brighton, N. Y.
Charles Adelbert Morse, Fort Madison, Ia.
Eugene Augustus Hoffman Tays, San José de Gracia, Sinaloa, Mexico.

As Associate Members.

Charles Aldo Alderman, Eau Claire, Wis.
Justin Burns, New York City.
Frank Taylor Chambers, Navy Yard, N. Y.
George Hallett Clark, New York City.
Carleton Emerson Davis, North Rochester, Mass.
Daniel Livermore Mott, Sangerfield, N. Y.
Charles Alfred Paquette, Indianapolis, Ind.
Henry Bedinger Rust, Pittsburg, Pa.
William Edward Schenck Strong, Detroit, Mich.
Harry Esmond Warrington, Cincinnati, O.

The Secretary announced the election by the Board of Direction on April 5th, 1898, of the following candidates:

As Associate.

Oliver Hazard Perry La Farge, New York City.

As Juniors.

Perry Robinson McNeille, Orange, N. J.
William Kerper Runyon, Newark, N. J.

The Secretary announced the death of John Bridgford, elected Fellow January 14th, 1871; died March 8th, 1898.

Adjourned.

April 20th, 1898.—The meeting was called to order at 20.30 o'clock, President Alphonse Fteley in the chair; Charles Warren Hunt, Secretary, and present, also, 86 members and 16 guests.

A paper by James Ritchie, M. Am. Soc. C. E., entitled "The Construction of the Lorain Dry Dock and Shipyard of the Cleveland Ship-Building Company," was presented by the Secretary and discussed by Messrs. L. L. Buck, F. Collingwood, Charles Macdonald, S. Whinery P. W. Henry and L. R. Pomeroy.

The Secretary announced the death of R. R. Minturn, elected Member November 2d, 1887; died April 7th, 1898.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

April 5th, 1898.—President Fteley in the chair, Chas. Warren Hunt, Secretary, and present, also, Messrs. Deyo, Just, Manley, Owen, Parsons, See and Wisner.

The territory occupied by the membership of the Society was divided into seven geographical districts, as required by Art. VII, Sec. 1, of the Constitution (see under "Announcements," below).

The action taken by the Board, on April 6th, 1897, in accepting the resignation of Horatio Seymour, was reconsidered, and at his request Mr. Seymour was reinstated as a Member of the Society.

Applications were considered and other routine business transacted.

One candidate was elected as Associate and two as Juniors.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society will be open every day hereafter from 9 to 22 o'clock, except on Sundays, when the hours will be from 14 to 19 o'clock.

NOMINATING COMMITTEE.

Under Art. VII, Sec. 1, of the Constitution, the Board of Direction has divided the territory occupied by the membership into seven geographical districts for the purposes of the Nominating Committee, and now announce this division to the membership:

District No. 1.—The territory within 50 miles of the Post Office in the City of New York.

District No. 2.—The States of Maine, New Hampshire, Vermont, Massachusetts, Rhode Island and Connecticut (except as included in District No. 1), and all countries in Europe.

District No. 3.—The States of New York and New Jersey (except as included in District No. 1), and the Dominion of Canada.

District No. 4.—The States of Pennsylvania, Delaware and Maryland, and the District of Columbia.

District No. 5.—The States of Ohio, Indiana, Illinois, Michigan, Wisconsin, Iowa and Minnesota.

District No. 6.—The States of Virginia, West Virginia, North Carolina, South Carolina, Georgia, Florida, Kentucky, Tennessee, Alabama, Mississippi, Missouri, Arkansas and Louisiana, the West India Islands, and all countries in South America.

District No. 7.—The States of North Dakota, South Dakota, Nebraska, Kansas, Texas, Montana, Wyoming, Colorado, Idaho, Utah, Washington, Oregon, California and Nevada; the following territories—Indian Territory, Oklahoma, New Mexico, Arizona and Alaska; the Republic of Mexico, and all countries in Central America, Asia, Australasia and Africa.

COLLINGWOOD PRIZE FOR JUNIORS.

This prize, which consists of $50 in cash, with an engraved certificate signed by the President and Secretary of the Society, was instituted and endowed in 1894.

The expectation of the donor, Francis Collingwood, M. Am. Soc. C. E., in restricting its award to papers prepared by Juniors, was that it would encourage the younger members to add to the permanent record whatever of interest came to their attention in their professional work.

This expectation has not been realized, and the reason for this failure is not easy to find. Nearly all who enter the Society in the grade of Junior do so shortly after graduation, and must, therefore, practice for at least four years before advancing to a permanent grade. It would seem, in view of these facts, that if the proper amount of effort were made, the result should be that each year there should be several papers presented by Juniors, worthy of the recognition which the award of this prize implies.

A simple, clearly expressed recital of problems encountered and overcome, or even a record of unsuccessful effort leading up to a discussion which throws light on the subject, will often prove of great interest.

It may be mentioned that during the past year the "Institution of Civil Engineers" awarded seven prizes to "Students."

It is hoped that this note will result in awakening the professional interest in the work of the Society, which should animate Juniors as well as Members in other grades.

ANNUAL CONVENTION.

The Thirtieth Annual Convention of the Society will be held in Detroit, Mich., during the last week of July, 1898. Messrs. George Y. Wisner, J. J. McVean and Chas. Warren Hunt have been appointed a

Committee of the Board of Direction to make the necessary arrangements.

Papers intended for presentation to the Convention must, in order to be issued in advance, be printed in the May number of *Proceedings.* Such papers should therefore be in the hands of the Secretary at an early day, as the date of issue of this number is May 25th, 1898.

MEETINGS.

Wednesday, May 4th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by H. N. Ogden, Jun. Am. Soc. C. E., entitled, "Flushing In Pipe Sewers," will be presented. It was printed in the March number of *Proceedings.*

Wednesday, May 18th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by R. S. Buck, M. Am. Soc. C. E., entitled, "The Niagara Railway Arch," will be presented. It is printed in this number of *Proceedings.*

Wednesday, June 1st, 1898, at 20 o'clock, a regular meeting will be held, at which Robert B. Stanton, M. Am. Soc. C. E., will address the Society on "The Cliff Dwellers." The address will be illustrated by the stereopticon.

Wednesday, June 15th, 1898, at 20 o'clock, a regular meeting will be held, at which a paper by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., entitled, "Origin of the Gulf Stream and Circulation of the Waters in the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," will be presented. It is printed in this number of *Proceedings.*

DISCUSSIONS.

Discussion on the paper by B. F. Thomas, M. Am. Soc. C. E., entitled, "Movable Dams," which was presented at the meeting of March 16th, 1898, will be closed May 1st, 1898.

Discussion on the paper by George Hill, Assoc. M. Am. Soc. C. E., entitled, "Steel Concrete Construction," which was presented at the meeting of April 6th, 1898, will be closed May 15th, 1898.

Discussion on the paper by James Ritchie, M. Am. Soc. C. E., entitled, "The Construction of the Lorain Dry Dock and Shipyard of the Cleveland Ship-Building Company," which was presented at the meeting of April 20th, 1898, will be closed June 1st, 1898.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.

RICHARD SUTTON BUCK	Niagara Falls, N. Y.	Assoc. M.	April 5, 1893
		M.	March 2, 1898
HARRY MONMOUTH HERBERT	Chf. Eng., New York and Philadelphia Traction Co., Bound Brook, N. J.		April 6, 1898
ARCHIBALD OLIN POWELL	St. Paul, Minn.		March 2, 1898
EDGAR KENNETH SMOOT	No. 7, Espiritu Santo, Ciudad de Mexico		Feb. 2, 1898
AARON TWYMAN	137 Watt Ave., Pullman, Ill.		March 2, 1898

ASSOCIATE MEMBERS.

FRANK TAYLOR CHAMBERS	Civil Eng., U. S. Navy, Navy Yard, Brooklyn, N. Y.	April 6, 1898
GEORGE HALLETT CLARK	406 West End Ave., New York City	April 6, 1898
PERCY HOLBROOK	56 West 47th St., New York City	March 2, 1898
HENRY BEDINGER RUST	5955 Alder St., Pittsburg, Pa.	April 6, 1898
DANIEL LAWRENCE TURNER	Lawrence Scientific School, Cambridge, Mass	Feb. 2, 1898
HARRY ESMOND WARRINGTON	Asst. Eng., C. N. O. & T. P. Ry. Co., 206 Odd Fellows' Temple, Cincinnati, Ohio	April 6, 1898

JUNIOR.

PERRY ROBINSON MCNEILLE	Orange, N. J.	April 5, 1898

CHANGES AND CORRECTIONS.

MEMBERS.

ARTHUR LINCOLN ADAMS	513 Stimson Bldg., Los Angeles, Cal.
DANIEL MARSHALL ANDREWS	U. S. Asst. Engineer, Lincoln, Ala.
WILLIAM ETHELBERT BELKNAP	35 Washington Ave., Wallabout Market, Brooklyn, N. Y.
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JOHN BRUNNER	6200 Alder St., Pittsburg, Pa.
WILLIAM MCLAURINE HALL	U. S. Engr. Office, Newport, R. I.

JOHN HISLOP .... First Asst. Engr., White Pass and Yukon Ry., Skaguay, Alaska.
WILLIAM BELL WHITE HOWE...... Spartanburg, S. C.
GEORGE HERBERT LELAND......... 929 Banigan Bldg., Providence, R. I.
JAMES DAVID MOFFET............. Gen. Roadmaster, Rio Grande Western Ry., Salt Lake City, Utah.
CHARLES HAYWARD MYERS......... 101 West 78th St., New York City.
JOHN CURTIS PATTERSON .... Chf. Eng., P. & E. R. R. Co., 1301 Land Title Bldg., Philadelphia, Pa.
THOMAS PETTIGREW.............. Chf. Eng., West Virginia Short Line R. R. Co., Clarksburg, W. Va.
JOSEPH LESLIE PHILLIPS.......... P. O. Box 607, El Paso, Tex.
IRA ALEXANDER SHALER........... 47 West 43d St., New York City.
ROBERT BREWSTER STANTON....... Sewickley, Pa.
EUGENE WASHINGTON STERN...... 56 West 72d St., New York City.
RUSSELL THAYER.................. Welsbach Street Lighting Co. of America, 246 North Broad St., Philadelphia, Pa.
BENJAMIN THOMPSON............ Asst. Eng., Southern Ry., Highland Park, Hamilton Co., Tenn.
LEE TREADWELL................. Care of Sooysmith & Co., Cornwall, Ont., Canada.
NISBET WINGFIELD............... Commr. of Public Works, Augusta, Ga.
MORGAN EDWARD YEATMAN........ Chiefden, Eltham, Kent, England.

ASSOCIATE MEMBERS.

EDWARD PONTANY BUTTS.......... 49 Pearl St., Hartford, Conn.
EDWARD HANSON CONNOR........ Alton, Ill.
HENRY STILSON FARQUHAR........ 874 Broadway, New York City.
HERBERT THOMAS GRANTHAM ..... 1535 North 33d St., Philadelphia, Pa.
JOHN FILLMORE HAYFORD......... 1514 Howard Ave., Washington, D. C.
WILLIAM WILLARD LOCKE......... 49 Concord St., South Framingham, Mass.
GEORGE ALFRED LUND............ 18 Buckingham St., Hartford, Conn.
GERDT HENRIK VEDELER.......... 2071 Madison Ave., New York City.
ALBERT LOWRY WEBSTER........ Civil and Sanitary Engineer, Drexel Bldg., 3 Broad St., New York City.

JUNIORS.

WILLIAM PARSONS BORIGHT........ Chatham, N. Y.
GEORGE HALLETT CLARK.......... 400 West End Ave., New York City.
JOHN A. CLARK, Jr............... 26 Front St., Newark, N. J.
CHARLES PERKINS COGSWELL, Jr... N. Y., N. H. & H. R. R., Buzzards Bay, Mass.
JOHN THOMPSON EASTWOOD........ Asst. Eng., Natl. Contracting Co., 1528 Polyhymnia St., New Orleans, La.
JOHN MAURICE EVANS. .... 164 West 128th St., New York City.
GEORGE GILL HONNESS............ 107 Washington St., Paterson, N. J.
FREDERIC ARNOLD KUMMER....... Sales Agt., Eastern Paving Brick Co., 701 Bowling Green Bldg., New York City.

Benjamin Franklin Latting......143 Liberty St., Room 909, New York City.
Henry Blakeman Lewis..........30 Gold St., New York City.
Abraham Bernard Samuelson....48 West 119th St., New York City.
Nelson Bowman Sweitzer, Jr.....1628 19th St., N. W., Washington, D. C.
Benjamin Franklin Welton.....280 Broadway, Room 115, New York City.

DEATHS.

John Bridgford ....Elected Fellow, Jan. 14th, 1871; died March 8th, 1898.
Roland Robinson Minturn.......Elected Member, Nov. 2d, 1887; died April 7th, 1898.
William Stark Rosecrans.......Elected Member, July 5th, 1882; died March 11th, 1898.

ADDITIONS TO

LIBRARY AND MUSEUM.

From Horace Andrews, Albany, N. Y.:
Annual Report of the City Engineer of Albany, N. Y., for the year ending January 1st, 1898.

From the Association of Engineers, Barcelona:
Revista Tecnológico Industrial. Vols. 17 to 20. [1894 to '97.]

From the Board of Water Commissioners, St. Paul, Minn.:
Sixteenth Annual Report. 1898.

From William H. Burr, New York City:
Elasticity and Resistance of the Materials of Engineering.

From F. W. Cappelen, Minneapolis, Minn.:
Annual Report of the City Engineer of the City of Minneapolis, Minn., for the year ending December 31st, 1897. Two copies.

From L. G. Carpenter, Fort Collins, Colo.:
Tenth Annual Report of the Agricultural Experiment Station of Colorado for the year 1897.

From W. H. Carruth, Lawrence, Kan.:
Kansas University Quarterly. Series A: Science and Mathematics.

From the Christian Science Reading Rooms, New York City:
Science and Health. Mary Baker G. Eddy.

From Otis F. Clapp, Providence, R. I.:
Annual Report of the City Engineer for the year 1897.

From the Commission des Annales des Ponts et Chaussées, Paris:
Five copies of "Etude d'une Nouvelle Formule pour calculer le débit des Canaux Découverts." M. H. Bazin, 1898.

From Alston Ellis, Fort Collins, Colo.:
Eighteenth Annual Report of the State Board of Agriculture and the State Agricultural College, including the Ninth Annual Report of the Agricultural Experiment Station, Fort Collins, Colo., 1896.

From Mordecai T. Endicott, Washington, D. C.:
Report of the Amor Factory Board, with accompanying documents.

From the Engineering News Publishing Company, New York City:
Manual of Statistics: the Stock Exchange Hand-Book for Investors and Stock Operators. Vol. XI, 1889.
Annual Reports of the City of Somerville, Mass., from 1880 to 1888.
Semi-Annual Report of City Comptroller of the City of Kansas, Mo., to December 31st, 1885: together with Reports of the City Engineer and Other Department Officers, 1885. Reports of the Comptroller, City Engineer and Other Officers of the City of Kansas, Mo., for 1883; and the Report of the City Engineer for 1884.
Annual Report of the City Engineer of Manchester, N. H., for 1886, 1888 to 1889, and 1895.
Annual Reports of the City Engineer of the City of Haverhill, Mass., for 1890 to 1895.
Annual Report of the City Engineer for Lawrence, Mass., 1889.
Annual Reports of the Town of Lenox, Mass., 1892.
Annual Report of the City Engineer of the City of Lynn, Mass., for 1888 to 1894.

Report of the City Engineer of the City of Marlborough, Mass., for 1892.
Report of the Superintendent of Streets of the Town of Medford, Mass., for 1892, and 1894.
Annual Report of the City Engineer of the City of Nashua, N. H., for 1891.
Annual Report of the Officers of the Town of Enosburgh, Vt., for 1890.
Census of Massachusetts for 1885. Vol. III.
Catalogue of Charles D. Miller & Son, Water Works Supplies, Utica, N. Y., for 1894.
Report of the General Superintendent of the Railway Mail Service for 1891 to 1893.
Annual Reports of the City Engineer of the City of Somerville, Mass., for 1890 and 1891.
Annual Reports of the City of Rockland, Me., 1889 to 1894.
Annual Reports of the City Civil Engineer of Portland, Me., 1890 to 1894.
Annual Reports of the Superintendent of Streets and Sewers of the City of Springfield, Mass., for 1884 and 1889.
Reports of the City Engineer of Toronto for 1882 to 1889.
Report of the City Engineer of Concord, N. H., for 1893.
Annual Reports of the Town of Laconia, N. H., for 1893 and 1894.
Annual Reports of the Board of Regents of the Smithsonian Institution, July, 1885, Part I, and July, 1894.
Annual Reports and Statements of the Chief of the Bureau of Statistics on the Foreign Commerce and Navgiation, Immigration and Tonnage of the United States for the years 1891 and 1892.
Biennial Report of the President of Fire and Police Commissioners of Memphis to the Governor of the State, December 1st, 1886.
Report of the City Surveyor of Quebec for the year 1872-73.
Commercial Relations of the United States with Foreign Countries during the years 1894 and 1895. Vol. I.
Ninth Annual Report of the Board of Directors of the Norfolk and Western Railroad Company, for the year ending December 31st, 1889.
Register of the University of California, 1895-96.
United States Trade-Mark Association Bulletin, September, 1893.
Catalogue of the State University of Iowa, 1895-96.
Annual Reports of the Directors to the Stockholders of the Chicago, Rock Island and Pacific Railway Company, 1891-94.
Proceedings of the Annual Conventions of the Roadmasters' Association of America, for 1883, 1884, 1887 to 90, and 1890 to '96.
Report of the President and Managers to the Stockholders of the Philadelphia and Reading Railroad Company, November 30th, 1891.
Forty-second Annual Report of the Managers of the Philadelphia and Erie Railroad Company to the Stockholders, February 14th, 1893.
Annual Reports of the President and Directors of the Canada Southern Railway Company for the years ending December 31st, 1892, 1893 and 1895.
Annual Reports of the Chicago and North Western Railway Company for the years 1890, 1892, 1893 and 1895.
Annual Reports of the Chicago, Milwaukee and St. Paul Railway Company for the years 1890-91, '93-95.
Annual Reports of the Officers of the Town of St. Albans, for the years ending February, 1882 and 1884 to '92.
Annual Reports of the St. Paul, Minneapolis and Manitoba Railway Company, for the fiscal years ending June 30th, 1892, 1893 and 1895.
Annual Report of the Chicago, Burlington and Quincy Railroad Company, for the years ending December 31st, 1889 and 1890.
Bids or Proposals for Building Sections A and B of the Croton Aqueduct in the Twenty-fourth Ward, New York City, 1884.
Specifications for the Construction of the Extension of the Sixty-eighth Street Tunnel, Chicago, 1892.
Two Copies of the Reports of the Aqueduct Commission of the New Croton Aqueduct, New York, 1884.
Proposal for Building Section 5 of the Sudbury River Conduit, 1875.
Proposal, Contract and Specifications for Constructing Fort Edward Water Works, 1893.
Quantities, Specifications, Proposals and Contracts for the Theresa Water Works, Theresa, N. Y.
Specifications for Buildings for the Greenville Water and Electric Light Co., at Greenville, Texas.
General Contract for the Greenville Water and Electric Light Co., at Greenville, Ohio.
Specifications for Water Works, Berryville, Va., 1891.
Specifications for Water Works, Red Bank, New Jersey, 1884.
Specifications, Forms for Proposals, and Articles of Agreement for the Astoria Water Works, Astoria, Ore., 1895.
Specifications for Water Works, Sault Ste. Marie, Mich., 1885
Specifications for the Water Tower and Intake for the Sault Ste. Marie Water Works, Sault Ste. Marie, Mich.
Contracts and Specifications for the Water Works of the Hackensack Water Co., 1881.
Specifications of the Milton Water Co., Milton, Pa., 1883.
Proposals for the Water Works of Waterbury, Vt., 1896.
Proposals for Riveted Steel Pipes for the Water Works of Duluth, Minn., 1896.
Specifications for a System of Water Works for Bethany, Mo., 1894.
Contract for the Gilbertsville Water Works, Gilbertsville, N. Y., 1896.
Contracts, Specifications, and Proposal for the Boonville Water Works, Boonville, N. Y., 1897.
Annual Reports of the Denver and Rio Grande Railroad Co. for the Years ending June 30th, 1891 to '95.
Proceedings of the New England Roadmasters' Association for 1883, 1885, 1886, 1888 to 1890, and 1892 to 1895.

Proceedings of the Association of Railway Superintendents of Bridges and Buildings, for 1893 to 1896.

Annual Reports of the Philadelphia and Reading Railroad Co., for the years ending November 30th, 1889 and 1890.

Annual Reports of the Northern Pacific Railroad Co. for 1891 to 1893.

Report of the Comptroller of the Northern Pacific Railroad Co., for the year ending June 30th, 1895.

St. Rollox Locomotive and Carriage Works of the Caledonia Railway. P. L. Dunn. 1897.

Treatise on Public Health and its Applications in Different European Countries. A. Palmberg. 1893.

Proceedings of the Meeting of the American Railway Association for 1896.

Returns of the Accidents and Casualties as Reported to the Board of Trade by the Several Railway Companies of the United Kingdom, 1894 to 1896.

General Report to the Board of Trade from the Railway Companies of the United Kingdom, for the year 1894.

Circulars from the Board of Trade to the Railway Companies of the United Kingdom and Correspondence Relative Thereto, 1895, 1896.

General Report to the Board of Trade upon the Accidents that have Occurred on the Railways of the United Kingdoms during the year 1894.

Administration Report on the Railways in India, for 1895-'96.

Returns by the Railway Companies in the United Kingdom on Signal Arrangements and Systems of Working, for 1893 to 1895.

Returns by the Railway Companies of the United Kingdom on the Workings of Continuous Brakes, 1894 to 1896.

Railway Returns for England and Wales, Scotland and Ireland, for 1894-1895.

Annual Reports of the Southern Pacific R. R. Company, for 1889-1892.

Annual Reports of the Directors of the Union Pacific Railway Companies, for the years ending December 31st, 1890 to 1893.

Annual Reports of the Louisville and Nashville Railroad Company, for 1890-91, 1891-92, 1892-93, 1894-95, and 1895-96.

Annual Reports of the Baltimore and Ohio Railroad, for 1890-1891.

Record of Transportation Lines of the Pennsylvania Railroad, for the years 1892, 1893, and 1895.

Annual Reports of the New York Central and Hudson River Railroad Company, for the years ended June 30th, 1890, 1891, and 1893 to 1895.

Report of the General Superintendent of Railway Mail Service, for the fiscal year ended June 30th, 1890.

Report on Cholera in Europe and India. E. O. Shakespeare, 1890.

Report of the Commissioner of Education for the year 1894-95, Vol. II.

Annual Report of the Commissioner of Railroads, State of Michigan, for the year 1891.

Annual Report of the State Board of Health of Massachusetts, for 1896.

Compendium of the Tenth U. S. Census, Part II.

Railroad Map of Wisconsin, 1886.

Atlas to accompany Report A C, Pennsylvania Second Geological Survey.

Vagaries of Sanitary Science. F. L. Dibble, 1893.

Official Railway List and Handbook of Useful Information for Railway Men, for 1894.

Annual Reports of the Board of Public Works of Kansas City for 1893 and 1894.

Annual Report of the City Engineer of the City of St. Joseph, Mo., for the year ending April 15th, 1889.

Annual Reports of the City Engineer of Omaha, Neb., for 1885, 1887 and 1890 to 1894.

Annual Reports of the Sewer Commissioner of St. Louis for the years ending April, 1890 to 1891.

Annual Report of the Board of Public Works of Kansas City, Mo., for 1889, and for the years ending April, 1891 and 1892.

Charter Amendments of Kansas City, Mo., Adopted February 27th, 1892.

Annual Report of the Board of Public Works of the City of Duluth, Minn., for the years ending February, 1888 to 1896.

Annual Report of the Engineering Department of the City of New Bedford, Mass., for the year 1894.

Annual Reports of the City Engineer of Newton, Mass., for the years 1883, 1884, 1886 to 1890, 1892 and 1895.

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BOOK NOTICES.

A TEXT BOOK ON ROOFS AND BRIDGES.

Part IV. Higher Structures. By Mansfield Merriman and Henry S. Jacoby. Cloth, 9 x 5½ ins., pp. 9 + 276. New York, John Wiley & Sons, 1898.

The first three parts of this work are devoted to framed structures having two supports whose reactions are vertical. In the present volume, which forms Part IV of the work, structures which have more than two supports, or which have two supports whose reactions are not vertical, are discussed. The investigations given are mainly those of the theory of stresses and their determinations by analytic or graphic methods.

The chapters are as follows: Continuous Bridges, Draw Bridges, Cantilever Bridges, Suspension Bridges, Three-Hinged Arches, Two-Hinged Arches, and Arches Without Hinges.

A number of diagrams serve to make more clear the description and analysis of the structures treated, and problems, designed to test the knowledge of the student, accompany each article.

WATER AND PUBLIC HEALTH.

The relative purity of water from different sources.

By James H. Fuertes, M. Am. Soc. C. E. Cloth, 7½ x 5 ins. 75 pp., 70 diagrams. John Wiley & Sons, New York. 1897.

The author's idea in this work has been to group the principal cities of the world into classes according to the quality of their public water supplies, and then to make a comparative study of their mortality statistics.

The descriptions of the sources of supply of American and European cities are taken from the author's private notes, from municipal reports, reports on improved supplies, etc. The statistics of typhoid fever, etc., are taken mostly from the published reports of the cities and of the State Boards of Health. Data which could not be obtained from these sources have been secured by correspondence.

The first chapter treats of the etiology and prophylaxis of typhoid fever, and its relations to rainfall, sewers, water supply, etc. Chapter II is entitled, "When Does Pure Water Pay?" Chapter III treats of the sanitary value of impounded and other water-supplies, and in the last chapter the author states the conclusions to be drawn from a study of the information and statistics given.

Appendix A shows the death-rates from typhoid fever in seventy-six cities, mostly American and European, during the six years, 1890-1895. Appendix B contains descriptions of the sources of the water supplies of seventy-five American and European cities. The annual rainfall for various American cities, from 1890 to 1895, inclusive, is given in Appendix C, and Appendix D is a table showing the number of times that different typhoid fever death-rates occurred in all the cities mentioned in Appendix A during the six years, 1890–1895.

The book contains a very complete index of places and subjects, and the diagrams illustrate graphically the ideas which the author has aimed to set forth.

1898. APRIL. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

CONTENTS:

THE NIAGARA RAILWAY ARCH.

By R. S. BUCK, M. Am. Soc. C. E.

TO BE PRESENTED MAY 18TH, 1898.

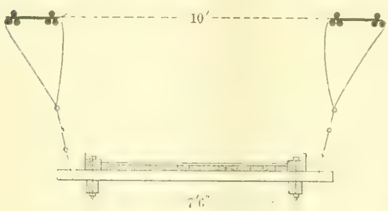
Historical.—Probably there is no bridge site on the Western Continent of greater technical as well as historic interest than that of the Niagara Railway Arch. Each of the former bridges at this site possessed in its day new and striking features, and marked a distinct advance in American engineering.

The plan of spanning the Niagara gorge with a suspension bridge probably first took practical shape when it was suggested to the Hon. William Hamilton Merritt, of St. Catharines, Ontario, by a description of the Freiburg Suspension Bridge in a letter from a friend. This was in 1844. In 1846, through Mr. Merritt's efforts, charters were obtained from the State of New York and the Canadian government for the construction of the first bridge across the gorge. The

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

scope of the bridge to be built was not then definitely determined, but the charters show an appreciation of the probable development of railroad facilities and the demand for a railroad bridge at this point. At that time there was no railroad to Niagara Falls from the West, although the Great Western, afterward a lessee of the bridge, was in course of construction.

First Suspension Bridge.—In the winter of 1847 the bridge companies made a contract with Chas. Ellet to construct a bridge on the site occupied by the present bridge. It was their ultimate purpose to build a railway bridge, but the plan was delayed for some years by the magnitude of the undertaking and lack of funds. Mr. Ellet first threw across the gorge a cable of thirty-six No. 9 wires, on which a light iron carriage was run for about a year and used for the purposes of the subsequent work and for passenger service. From this was developed the earliest bridge, shown in Plate VII, which was completed in 1848. This bridge had no stiffening truss. Its towers were of wood, and the expansion rollers consisted of a single wooden cylinder under each group of cables, that is, two cylinders on each tower. A cross-section of this bridge is shown in Fig. 1.



CROSS SECTION OF BRIDGE AT CENTER

Fig. 1.

Mr. Ellet's connection with the work ceased on the completion of this bridge, and he had no hand in planning the railway bridge as finally built in 1853–1855.

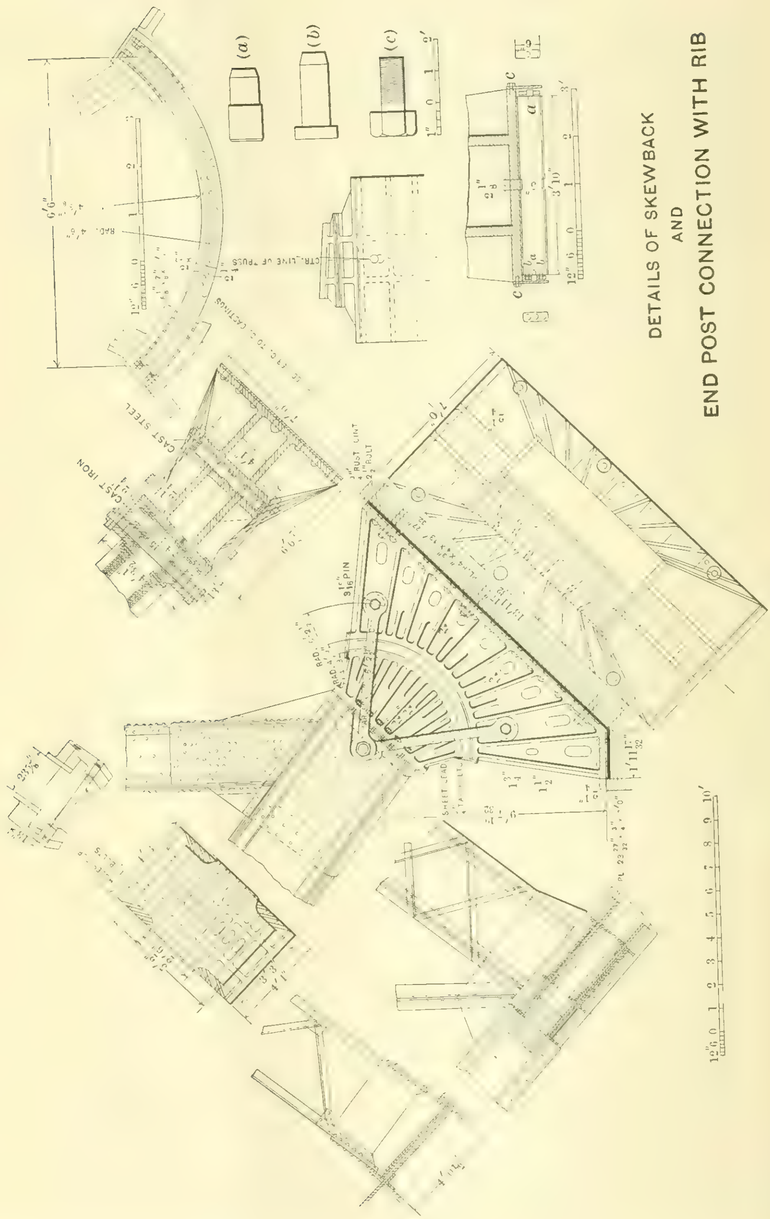
Railway Suspension Bridge.—The conception, development and execution of this bridge were the work of John A. Roebling. Both as an engineering feat and as an historical event, Mr. Roebling's great work is of enduring interest. It is fair to say that in the Niagara Railway Suspension Bridge the results of theoretical research were more successfully applied to practical conditions, so far as the strength of materials is concerned, than in any other bridge built, up to that time. A view of this bridge is shown in Plate VIII.

Prior to this, Mr. Roebling had built six suspension bridges, but these were for light highway traffic and did not demonstrate his abilities to the extent shown by this work.

The idea of a suspension bridge for railway service met with strong opposition, some of it from high sources. Its opponents insisted that a suspension bridge under the weight of a railway train must necessarily be subjected to excessive and dangerous deflection. Mr. Stevenson was at this time evolving the plans for the Victoria Tubular Bridge at Montreal, and opinion was divided as to the comparative merits of the two types. It was no small part of Mr. Roebling's task to overcome the prejudice against his chosen type of bridge. Even after its completion and successful operation for several years, it was still the object of much criticism, most of which was biased and absurd. It is a strange coincidence that these two bridges, the Niagara Railway Suspension Bridge and the Victoria Tubular Bridge, built at about the same time and for the same object, but so totally different in principle, should serve for almost the same length of time and pass out of existence together, to give place to more vigorous successors, better capable of meeting the ever-growing exactions of trade and travel.

It was Mr. Roebling's firm conviction that no other type of bridge was adaptable to the Niagara gorge, and that the suspension bridge was the coming type for long-span bridges. In the first view he was mistaken, for of the three bridges now spanning the gorge, only one is a suspension bridge, and that is being replaced. In the second, he was likewise wrong, and yet in a measure, right. The Niagara Bridge was the only railway suspension bridge ever built. Only one other was commenced, and that was not completed. Still this type has been accepted by high authority as available for spans of a length beyond the reach of other types, for railway as well as highway service. Even viewed in the light of increased experience, and among the vastly multiplied works of the engineer, despite flaws developed by long service, and reconstruction rendered necessary by time and abuse, the Niagara Railway Suspension Bridge will long remain a monument to engineering skill. It was a great leap toward the high plane occupied by bridge construction at the present day.

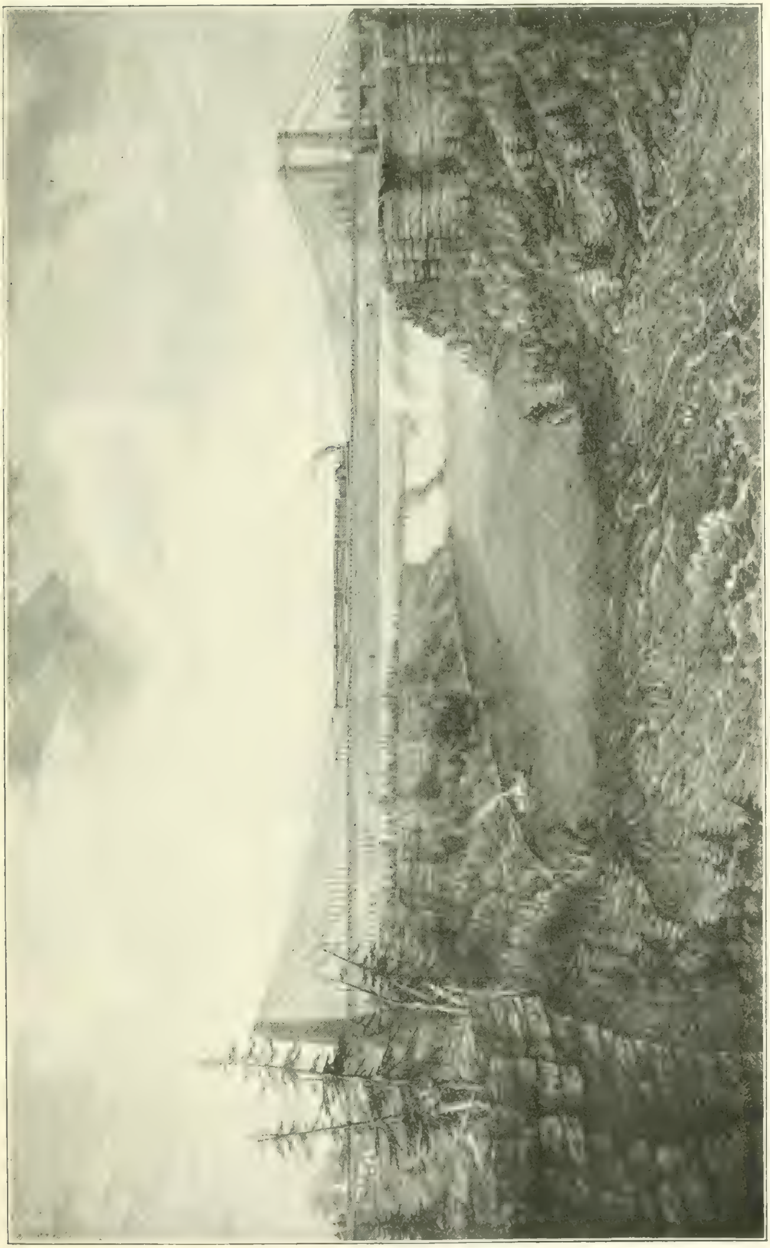
Reconstruction of Railway Suspension Bridge.—In 1877, examination disclosed that the outside layers of wires in the cables had corroded at the anchorages. The cables were here embedded in concrete. The strain on them due to moving loads had worked them loose from the concrete, and left a small surrounding space open to the admission of



DETAILS OF SKEWBACK AND END POST CONNECTION WITH RIB

Fig. 2.

PLATE VIII.
PAPERS AM. SOC. C. E.
APRIL, 1898.
BUCK ON NIAGARA RAILWAY ARCH.



water. This resulted in considerable corrosion, especially underneath the cables, from the face of the masonry back to the shoe.

The renovation of the cables and all the subsequent work of renewing the bridge was designed and executed by L. L. Buck, M. Am. Soc. C. E. The defective wires were cut out and the sound ends connected and spliced under proper stress. The greatest number of wires that required splicing at one end of any of the cables was 65. The wires removed were thoroughly tested to ascertain if there had been any deterioration other than that due to local corrosion. The results indicated none whatever. In fact, some of the wires, corroded partly through, showed a greater ultimate strength per square inch of remaining section than did the unaffected wire.

The wire in the cables of the early Ellet bridge was used in the cables of the railway bridge. The total number of wires in the four cables was 14 560. When the cables were taken down last year the wires were in an excellent state of preservation. In fact, it can be safely stated that, after 42 years of service, they were as sound as when first placed in position. It is interesting to note that when the strands were cut into short lengths they curled up, an indication that they still retained the set given them by the reels on which they had originally been coiled, and that they had not been overstrained.

During the work of repairing the cables it was discovered that parts of the anchorage had been badly strained, by reason of imperfectly formed eye-bar heads, light pins, and imperfect packing. To reinforce these, two new anchorages were put down behind the old ones, on each side of the river, connecting directly with the shoes, carrying the strands of the cables. This addition increased the strength of the anchorages about 50 per cent.

While reinforcing the anchorages, Mr. Buck made a careful study of the problem of renewing the stiffening truss. The old wooden truss was very badly decayed and racked, and was fast becoming ineffective. His plans contemplated replacing the wooden truss with a metal truss without interrupting traffic. At the time this was considered a very daring undertaking, and grave fears were felt as to its safety and success. However, in 1880 the entire plan was carried out without a single serious mishap. This change decreased the dead load on the cables by 178 tons, and permitted a safe increase of live load of from 200 to 350 tons.

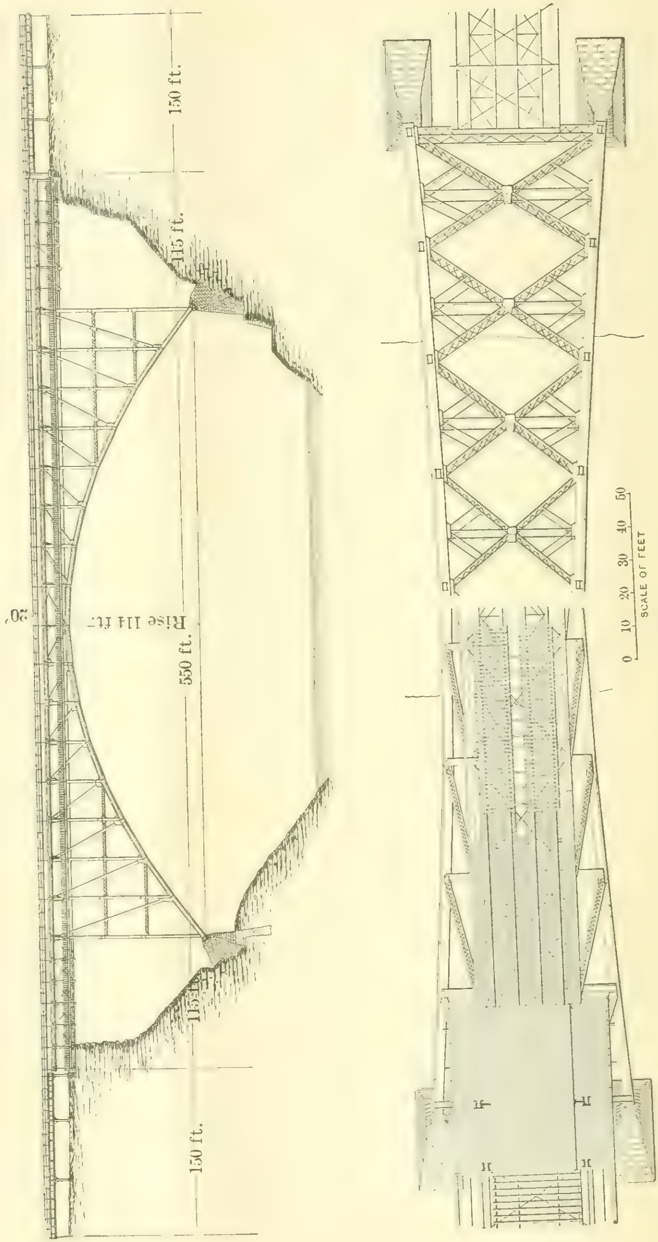

150 ft.
115 ft.
Rise 114 ft.
20'
550 ft.
115 ft.
150 ft.
0 10 20 30 40 50
SCALE OF FEET

FIG. 3.

PLATE IX.
PAPERS AM. SOC. C. E.
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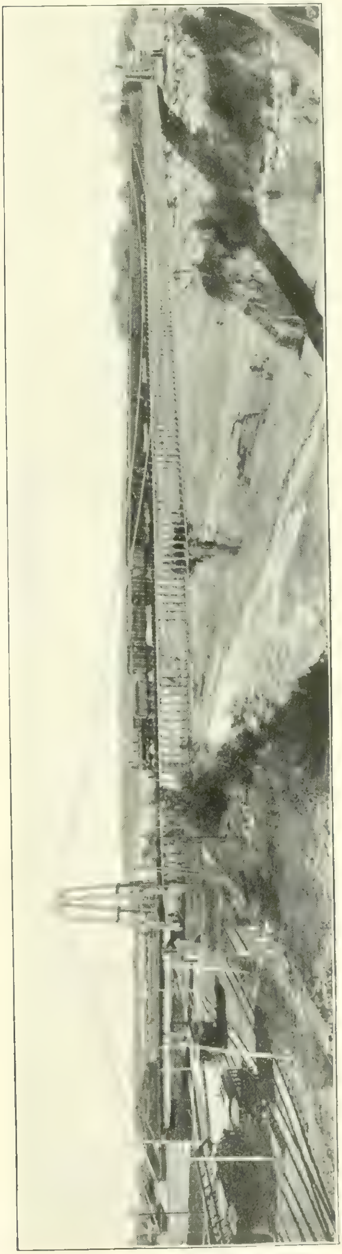
BUCK ON NIAGARA RAILWAY ARCH.



A full description of the work of reinforcing the anchorages and renewing the stiffening truss can be found in Mr. Buck's paper on the subject in the *Transactions* of the Society, June 15th, 1881.

In 1886 it was decided that safety demanded the renewal of the stone towers carrying the cables. These had for some years shown signs of disintegration, but they were kept in fair condition by replacing defective stones from time to time with sound ones. The disintegration was due to the inferior quality of the stone, and was augmented by the failure of the rollers under the cable saddles to perform their function because of rust. In fact, when the roller beds were taken out, the rollers were found to be fixed immovably in a mass of rust and cement, which had worked its way in from the mortar with which the saddles were originally covered by Mr. Roebling. This caused rocking of the towers under live load and changes of temperature, and greatly accelerated the destructive action of the frost on the masonry. It was at first attempted to preserve the towers by cutting away the defective surface stones and casing them in sound masonry, but it soon developed that the disintegration had penetrated too deeply to be remedied by this means. It was then decided to replace them with towers of iron.

Briefly described, this was accomplished as follows:

The corners of the stone towers were cut away, to admit the piers and legs of the new towers, which were then placed in position and temporarily secured to the former with clamps. The saddles carrying the cables on one tower were then lashed securely to a lifting frame, consisting of bent eye-bars and built beams, and the two cables were raised together by means of six 125-ton hydraulic jacks, resting on the new tower leg. When lifted high enough, the weight of the cables was taken on four short cast columns, one at the top of each tower leg. The old saddle bearings and three courses of masonry were then removed, and the heavy built base to take the bed plates under the saddles was moved into place. The new rollers and bed plates were then placed under the saddles and the weight taken on them. While the cables were being lifted, a period of about eight hours, no trains were allowed on the bridge. This completed the work of reconstruction of the suspension bridge. There then remained nothing of the original structure, except the cables, saddles, suspenders and anchorages. The reconstructed bridge is shown in Plate IX.

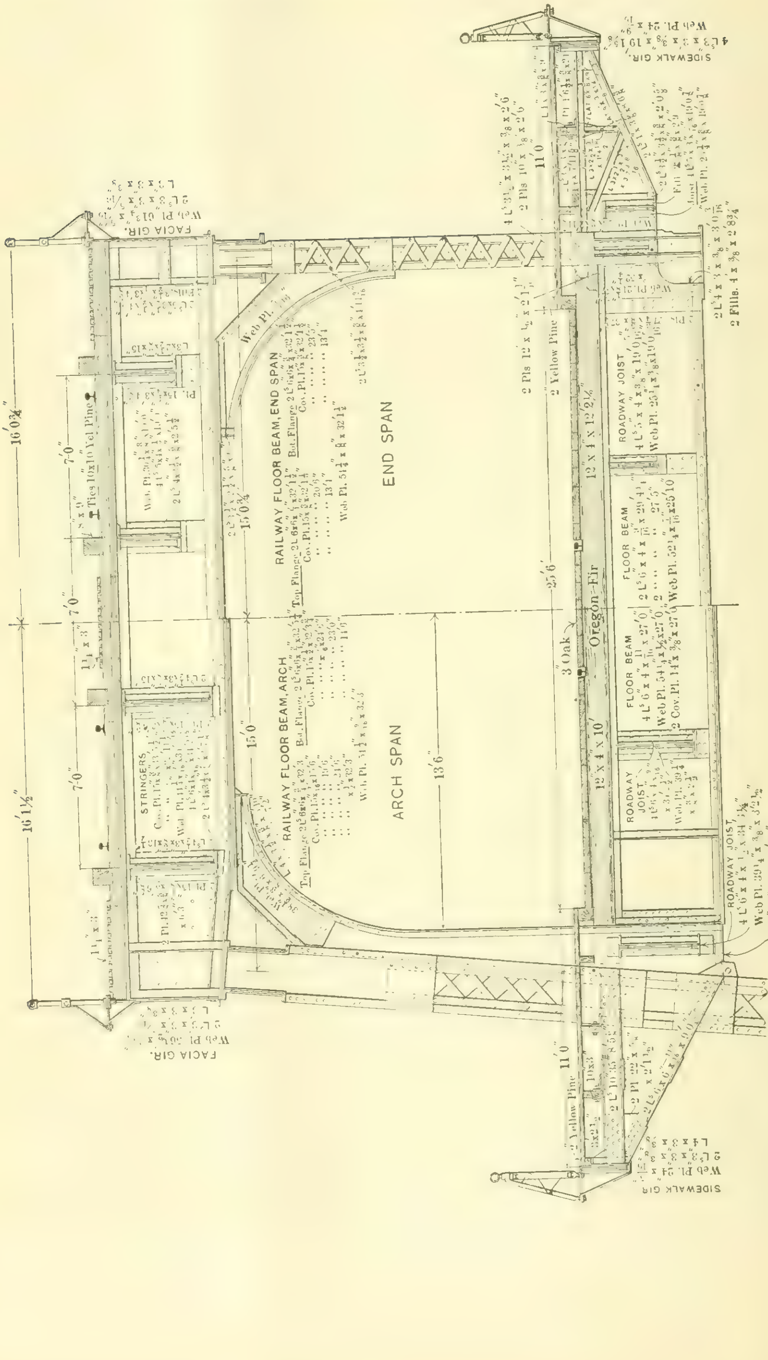

FACIA GIR.
FACIA GIR.
STRINGERS
RAILWAY FLOOR BEAM, END SPAN
RAILWAY FLOOR BEAM, ARCH
END SPAN
ARCH SPAN
FLOOR BEAM
FLOOR BEAM
ROADWAY JOIST
ROADWAY JOIST
ROADWAY JOIST
SIDEWALK GIR.
SIDEWALK GIR.
Oregon Fir
3″ Oak
2″ Yellow Pine
16′0¾″
16′1½″
25′6″
13′6″
15′0″
11′0″
11′0″

FIG. 4.

PLATE X.
PAPERS AM. SOC. C. E.
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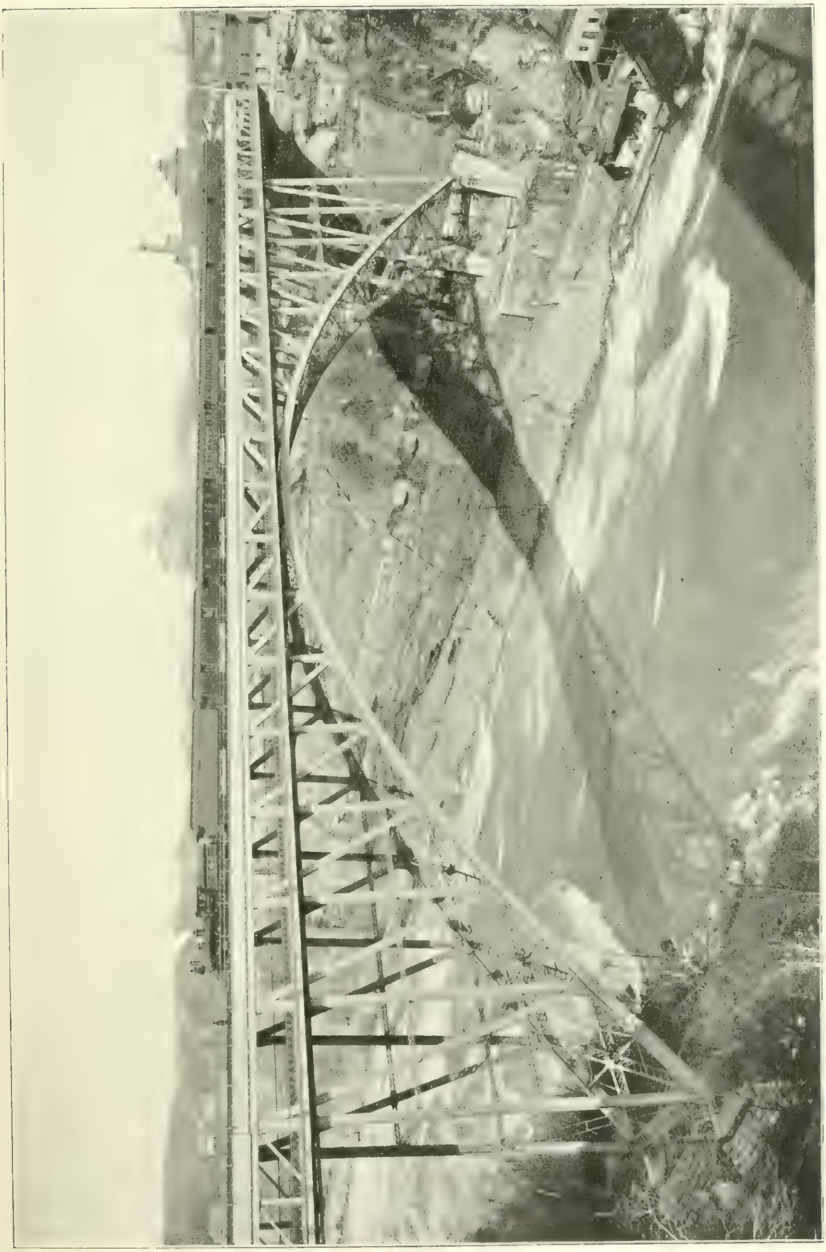
BUCK ON NIAGARA RAILWAY ARCH.



It was thought, when it was decided to replace the suspension bridge, that the old bridge could be utilized at another site, but when the work was done, no site was available; and, owing to the difficulty and expense of taking it down in proper condition for re-erection, the whole structure, except, perhaps, the suspenders and the wind-guys, was consigned to the scrap heap. Some of this material, after going through the furnace and rolls, will appear again in the Niagara Falls and Clifton Bridge.

Arch.—The completed arch is shown in Plate X. In treating of the present bridge, a simple recital of the facts, as observed by the author, is all that he can contribute towards a discussion of the principles involved in the evolution of a work with which he was fortunate in being associated, during its design and execution. He feels that much of value can be and should be contributed on the subject of steel arch construction from many well-equipped sources, and therefore hopes that whatever is lacking in the paper will be forthcoming in its discussion.

The steel arch has, within the past few years, grown greatly in interest and importance, and is entitled to full consideration. It is rigid, and, at such a site as the Niagara gorge, is economical beyond any other type. It also stands far ahead of all others, except, that in point of beauty, perhaps its anti-type, the suspension bridge, takes first place in the minds of some.

There is lacking the simple practical treatment of metal arches which has been given to other types of trusses, a treatment which would supply the wants of the engineer who seeks results and cannot afford to master the numerous partial and abstract treatises in order to reach them.

The fund of information on this subject is not scant, but it needs concentration.

The author has derived much assistance from Professor Chas. E. Greene's book on arches, but this does not supply the entire need.

Division of Types.—Arches are usually divided into three general types: 1st, three-hinged; 2d, two-hinged; 3d, hingeless or elastic. Without changing the form or arrangement, or in fact anything other than inconspicuous details, the same general design can be put in any of the three classes, and in each instance will be subjected to radically different stresses and deflections under load. The problem as to which

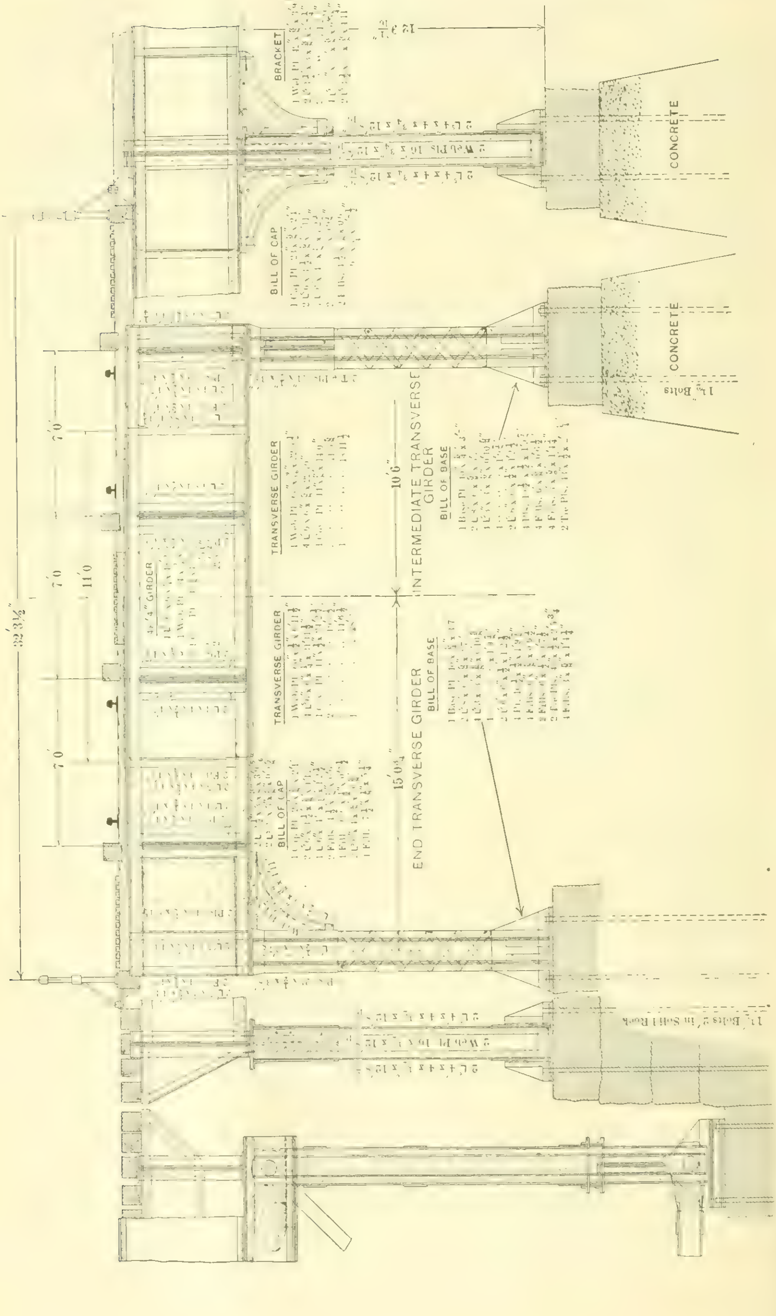

BRACKET
CONCRETE
CONCRETE
BILL OF CAP
TRANSVERSE GIRDER
TRANSVERSE GIRDER
INTERMEDIATE TRANSVERSE GIRDER
BILL OF BASE
END TRANSVERSE GIRDER
BILL OF BASE
BILL OF CAP

FIG. 5.

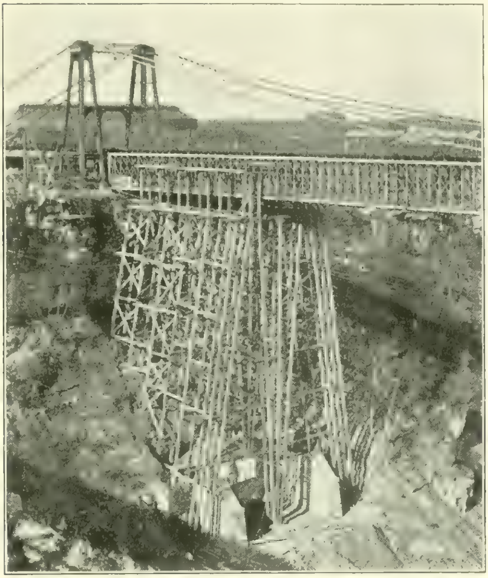


FIG. 1.

FIG. 2.

of the three types is best suited to given conditions is difficult to solve.

The three-hinged arch has been, and will perhaps continue to be, a popular type, on account of its simplicity of computation and adjustment, and the practical absence of temperature stresses. However, what is gained in these respects is at the cost of rigidity, a matter of smaller importance in roof trusses, but of great importance in bridges, especially in those for heavy service. Every hinge is intended to provide for movement, and facilitates distortion under eccentric loading.

A marked advantage in removing the center hinge is that reversal in the web members is greatly reduced, and the top chord is made to carry a larger proportion of the stresses which are otherwise carried almost entirely by the rib.

Hence the question arises: which is preferable, ease of calculation and adjustment, inconsiderable temperature stresses and greater vibration, or greater rigidity with increased temperature stresses and difficulty of adjustment? On similar grounds, comparison can be made between the two-hinged and the hingeless arches.

Niagara Railway Arch.—In the design of the Niagara Railway Arch the problems presented by the excessive loading to be provided for, by the length of the span and in the erection, which had to be accomplished without interruption to traffic, all required careful treatment.

The Chief Engineer, after a thorough investigation of all available types, fixed on the two-hinged spandrel-braced arch as best meeting all requirements. In 1882–83, when the subject of building a bridge for the Michigan Central Railway, across the Niagara Gorge, was under consideration, he prepared a design and estimate for a spandrel-braced arch for that work, to be erected in the same manner as the Niagara Railway Arch. This design included the center hinge. However, no opportunity was given to present it to the Bridge Company. The present cantilever was the design adopted and built. The Driving Park Avenue bridge at Rochester, also designed by him, was a three-hinged spandrel-braced arch. The cantilever method of erection was likewise contemplated in the Rochester design, but as the use of false-work there was not impossible, and the method of erection was optional with the contractors, they adhered to the latter method.

After a careful consideration of the vibrations of the Rochester bridge, under loads most calculated to produce vibrations, Mr. Buck

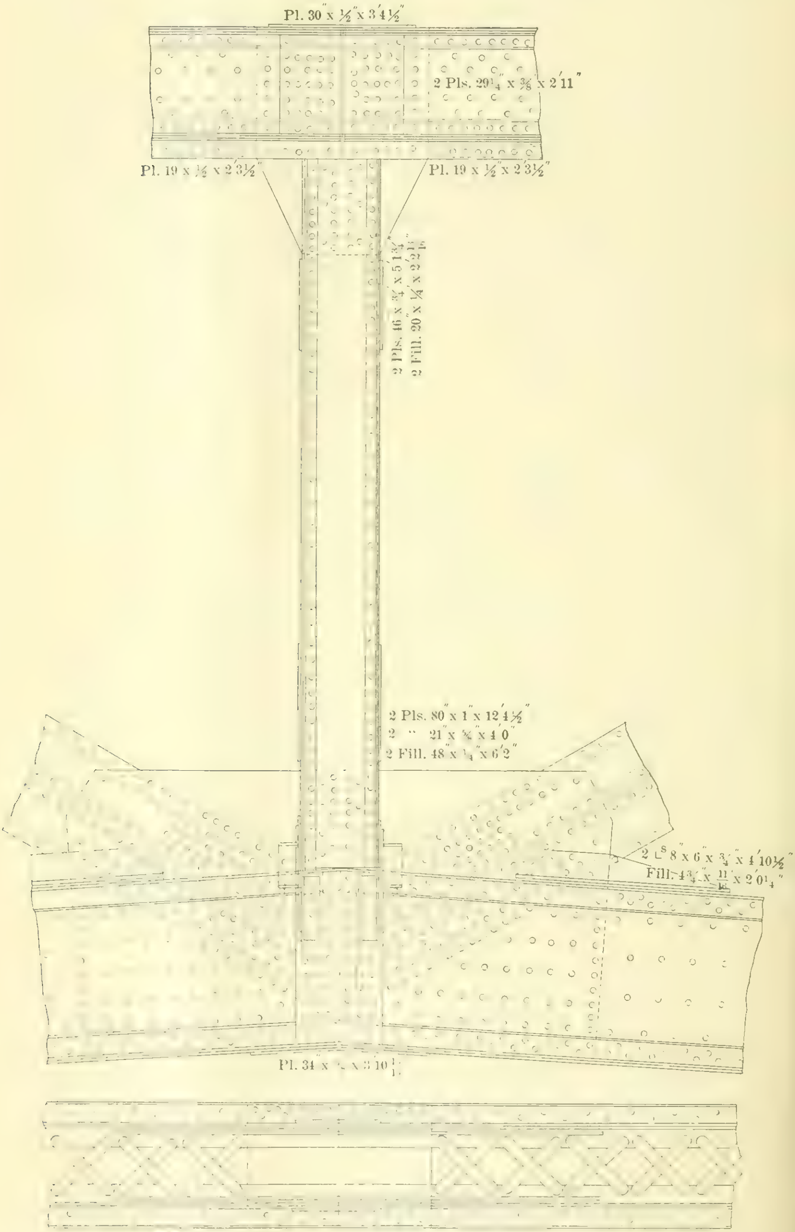

Pl. 30" x ½" x 3' 4½"
2 Pls. 29¼" x ⅜" x 2' 11"
Pl. 19 x ½ x 2' 3½"
Pl. 19 x ½" x 2' 3½"
2 Pls. 46" x ¾" x 5' 1¾"
2 Fill. 20" x ¼" x 2' 2"
2 Pls. 80" x 1" x 12' 4½"
2 " 21" x ¾" x 4' 0"
2 Fill. 48" x ¼" x 6' 2"
2 Ls 8" x 6" x ¾" x 4' 10½"
Fill. 4¾" x 11/16" x 2' 0¼"
Pl. 34"

FIG. 6.

PLATE X.
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decided that it would be best to omit the center hinge in the Niagara arch. Yet his investigations showed that at Rochester, as probably in most cases, the vibrations seemed to a person standing on the structure to be much greater than they actually were. The results in this regard in the Niagara Arch are very gratifying. Vibrations due to trains passing over at a rate of 20 miles per hour are scarcely noticeable, while the irresistible jog trot of a horse seldom produces the usual responsive swing. The stiffness thus attained, the author believes, has never been found in any other bridge of equal span. The calculated deflection under a moving load of 10 000 lbs. per running foot is $1\frac{5}{16}$ in., and the observed deflection under the test load, which was about 6 500 lbs. per running foot, was $\frac{13}{16}$ of an inch.

Method of Calculation.—The absence of the center hinge in the spandrel-braced arch renders the calculation of stresses decidedly more difficult than in the three-hinged type. The method of calculation used was that given in Professor Greene's book on Arches, Chapter XII. However, the sections of the rib in the Niagara Railway arch are increased so as to be a mean between those required by this method and what would be required if there were a third hinge. This was done to meet any inaccuracy of adjustment due to varying temperature.

Foundations.—The skewbacks of the arch span were located with a view to bring the thrust of the arch on the "Clinton Ledge," a solid stratum of gray limestone, from 12 to 14 ft. thick, about half way between the water and the top of the bluff. Above is a blue shale, and below is the beginning of the Medina sandstone formation, thin layers of shale and sandstone sometimes running into solid sandstone 4 to 5 ft. thick. The bearing comes very fairly on the ledge on the New York side, where the stone was cut at the right angle to receive the masonry directly. But on the Canada side, the bearing was not so favorable and concrete had to be used under the front of the south skewback and under the entire north skewback. The heavy face wall under the New York skewbacks was necessitated by the undue encroachment of the Gorge Road upon the site. The cut made for this road left here an almost vertical face, liable to disintegration on exposure, directly at the front of the skewbacks. The skewback masonry is limestone with granite copings, entirely of dimension stones with half-inch joints and strong bond.

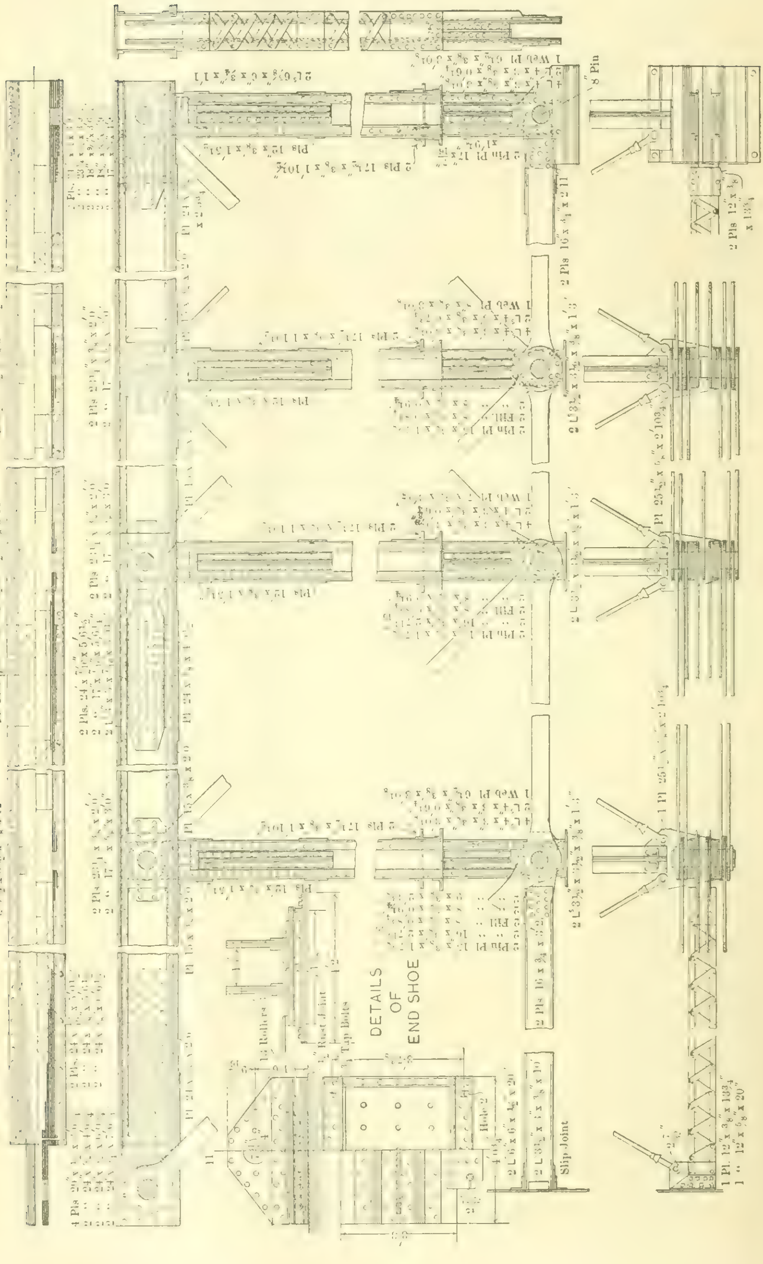


Fig. 7.

FIG. 1.

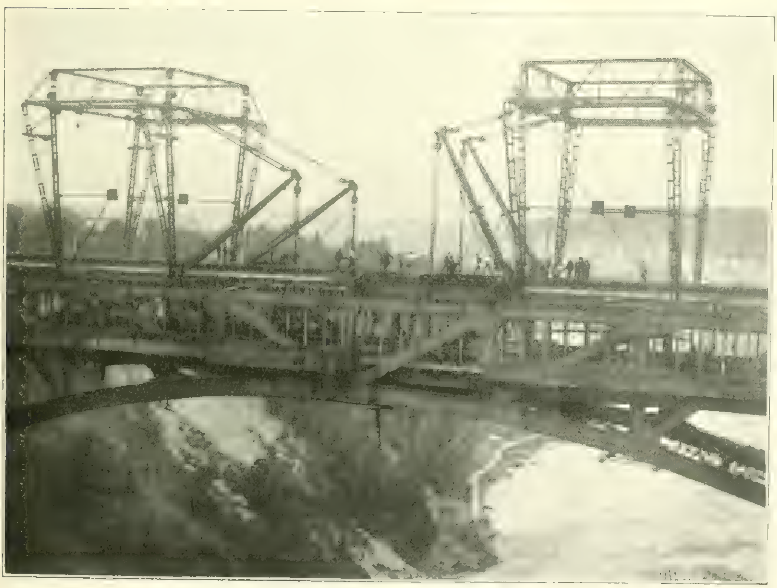


FIG. 2.

All masonry, except the retaining wall under the New York skew-backs, was put in with rented plant and hired labor, with results very satisfactory, both as to the cost and the quality of the work obtained.

The maximum loads on the masonry are as follows :

On top of coping..............	339	lbs.	per	square	inch.
Under "	300	"	"	"	"
On concrete....................	113	"	"	"	"

Rust Joint.—The rust joint, between the masonry and the shoe, is a mixture of 32 parts of cast-iron filings to 1 part of sal ammoniac by weight, very thoroughly rammed. These ingredients and proportions were adopted after experimenting with several formulæ. Thorough ramming is the most important part of the operation.

End Bearings.—The details of the end bearings of the arch span are shown in Fig. 2.

They consist of two steel castings, having between the concave face of the lower and the convex face of the upper a nest of 45 segmental rollers set radially with respect to the center movement at A. The axis of the cylindrical bearing faces is likewise at A and perpendicular to the vertical axial plane of the bridge. This form of bearing reduces frictional resistance much as a ball bearing does, and was adopted to avoid the use of an excessively large pin, with which, movement is rather doubtful of realization.

In placing the rollers, the outside plugs b were inserted temporarily, to hold them in their correct radial position and render them fixed. At the top the rollers almost touch each other, and in the wider spaces at the bottom are $\frac{5}{16}$-in. square bars to cause contact and restrict movement should there be any tendency to overturn. The bars, tap-bolted on both the upper and the lower castings at each end of the roller beds, are further safeguards against undue movement. Thus the rollers act like leaves, and can move either way through only a limited range without binding. As the movement of the rollers, due to moving load and temperature, is scarcely appreciable, there is no danger of the limit being reached. After the first panel of the arch was completed, connected with the anchorage, and swung back to correct position for proceeding with erection, the check plugs were removed and the rollers were thus freed. The center plugs and the guide bars remained permanently.

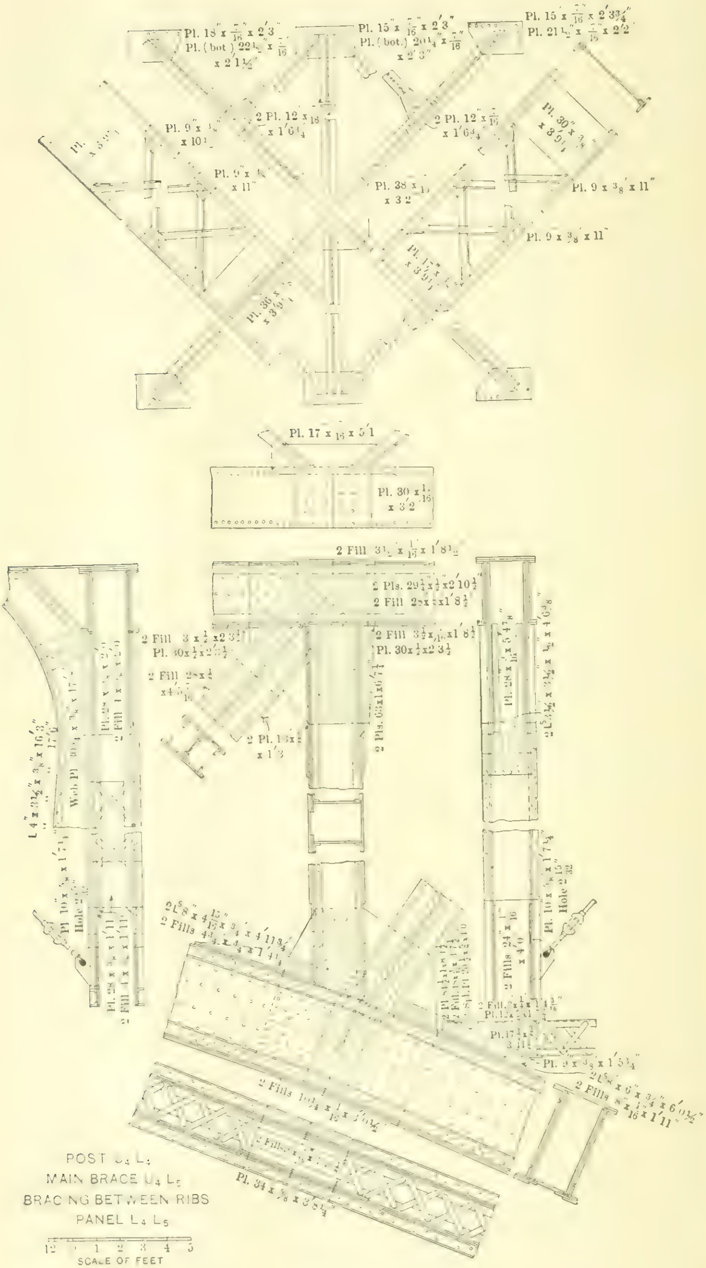

POST U4 L5
MAIN BRACE U4 L5
BRACING BETWEEN RIBS
PANEL L4 L5
SCALE OF FEET

FIG. 8.

The bearing on the rollers, with maximum load on the bridge, is 2 200 lbs. per lineal inch of roller, assuming the pressure to be uniform on all the rollers. The upper and the lower castings were cast each in one piece. The $\frac{3}{4}$-in. plates on the bottom of the lower castings were intended as a precaution against any possible rupture of the castings.

The manufacture of the bottom castings gave considerable trouble on account of their failure to shrink in the usual manner of steel castings, which was doubtless due to the thinness of the metal and the unyielding nature of the cores.

The eye-bars connecting the rib directly to the lower casting were intended to prevent any possible displacement of the rib or upper casting, a precaution needed probably only during erection.

Trusses.—The trusses, as stated above, are spandrel-braced, with horizontal top chord and parabolic rib. They are battered 1 in 10. The inclination of the planes of the trusses with reference to the end bearings is provided for by a double beveled face on top of the upper casting. This is the only double beveled face in the arch.

The camber of the arch was designed to be 8 ins. at 60° Fahr. It has been observed to range from 10 ins. at summer heat to 7 ins. at zero.

The arrangement and details of the arch span, as well as of the end spans and approaches, are shown in Figs. 3, 4, 5, 6, 7, 8 and 9, and need no further explanation here.

Erection.—The erection was an interesting feature of this work. One of the main objects in view was to maintain traffic, and this object was very fully accomplished. Not a single train was delayed, and traffic on the highway floor was suspended only for about two hours each day while the upper floor system was being put in, the time of day selected being that when there were the fewest trains. The lower floor was closed because of the danger to people passing below during the necessarily hurried operations of tearing out the old and putting in the new upper floor.

The deflection of the old bridge under moving load, the constant passing of trains, and the scant clearances at many points were considerations demanding close and constant attention; but as each anticipated difficulty was reached, it usually lost much of its formidable aspect. Besides, there was the comforting assurance that

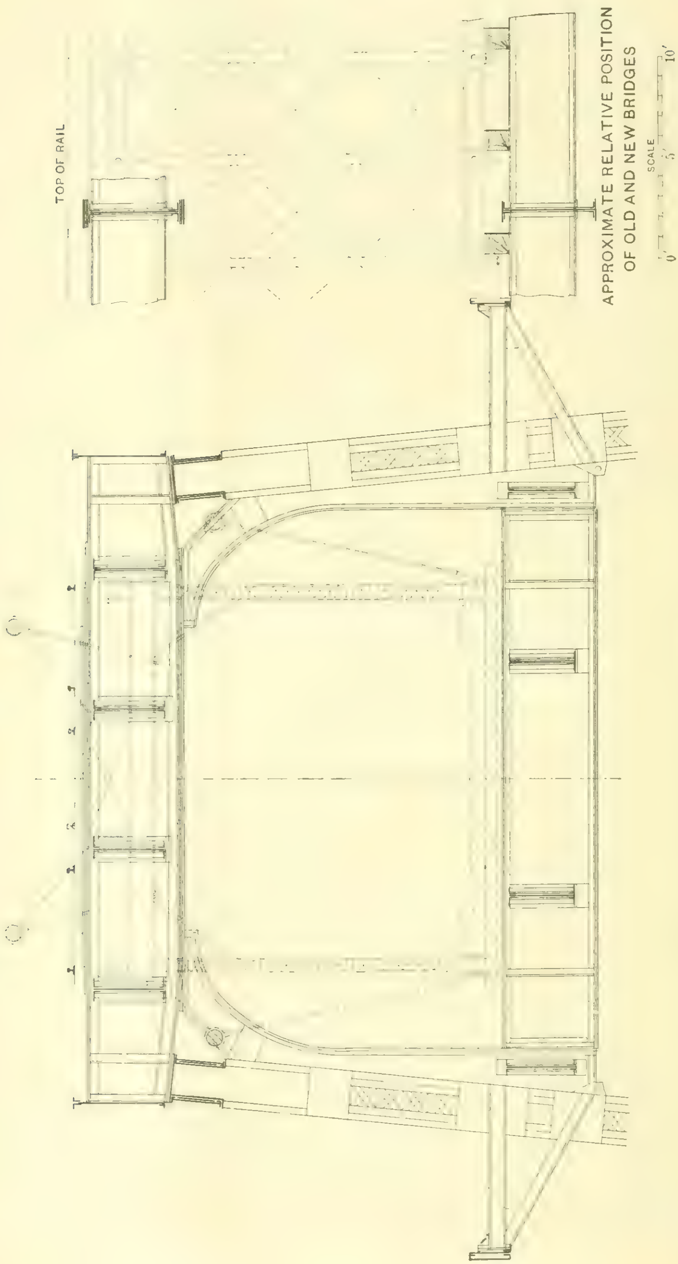

TOP OF RAIL
APPROXIMATE RELATIVE POSITION
OF OLD AND NEW BRIDGES
SCALE
0' 5' 10'

FIG. 9.

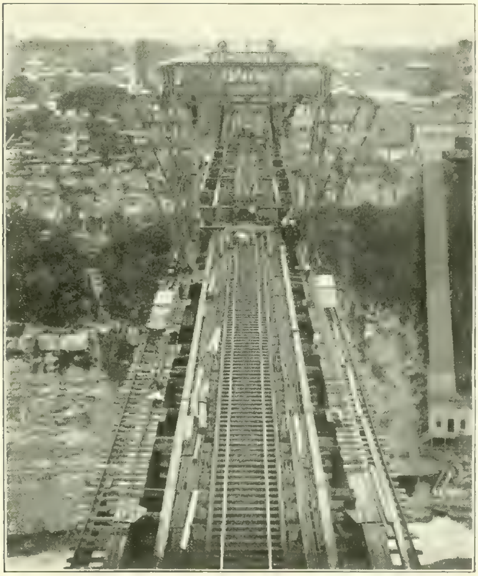


FIG. 1.

FIG. 2.

what had been accomplished with the old bridge under considerably less favorable circumstances ought to be accomplished again.

Briefly, the principle of erection was to build out the two halves of the arch as cantilevers anchored to the solid rock on top of the bluff, by means of adjustable anchor chains connecting with the arch at the top of the end post. The anchor chains consisted of the top chords of the end spans, such of the eye-bars of the end spans as were adaptable to the purpose, and such additional eye-bars and slabs as were necessary to complete the connections. The slabs were used as a matter of economy to serve as short eye-bars. The anchor chain was brought from vertical to horizontal by means of the "spider" shown in Fig. 10, which also shows the principal details of the anchor and the adjusting toggle. This is also shown in Fig. 1, on Plate XIII. The anchor pits were cut into the solid rock, back near the anchor walls of the old bridge. They are shafts 3 ft. x 6 ft. in section, and 19½ ft. deep. Chambers were excavated at the bottom of sufficient size to admit the anchors. These anchor pits had to be excavated with great care, to avoid shattering the surrounding rock, and the work was done by hired labor. Border holes were drilled as closely together as possible, to the full depth of the shaft, and the core was then blasted out with light charges of dynamite.

After the anchors and the first two sections of the anchorage chain were placed, the anchor pits were filled with concrete to the top of the rock. Although no provision was made to allow the eye-bars bedded in the concrete to sketch without interference with the concrete, no cracks appeared on its surface until six panels of the arch had been completed. Then some very slight cracks were observed at the corners of the outside bars, but these showed no increase as the work continued to the center.

Adjusting Toggle.—The principle of the adjusting toggle is not new, but its adaptation to this case was very effective. Its operation is apparent from the figure. The right and left screw was turned by hand with capstan bars. Some doubt was felt as to its ability to lift as well as to lower the load coming on it from the weight of the half spans. But this was done without difficulty, nineteen men working each screw.

Erecting Plant.—Under the plan of erection originally contemplated, material was to be conveyed and placed by means of cable ways sup-

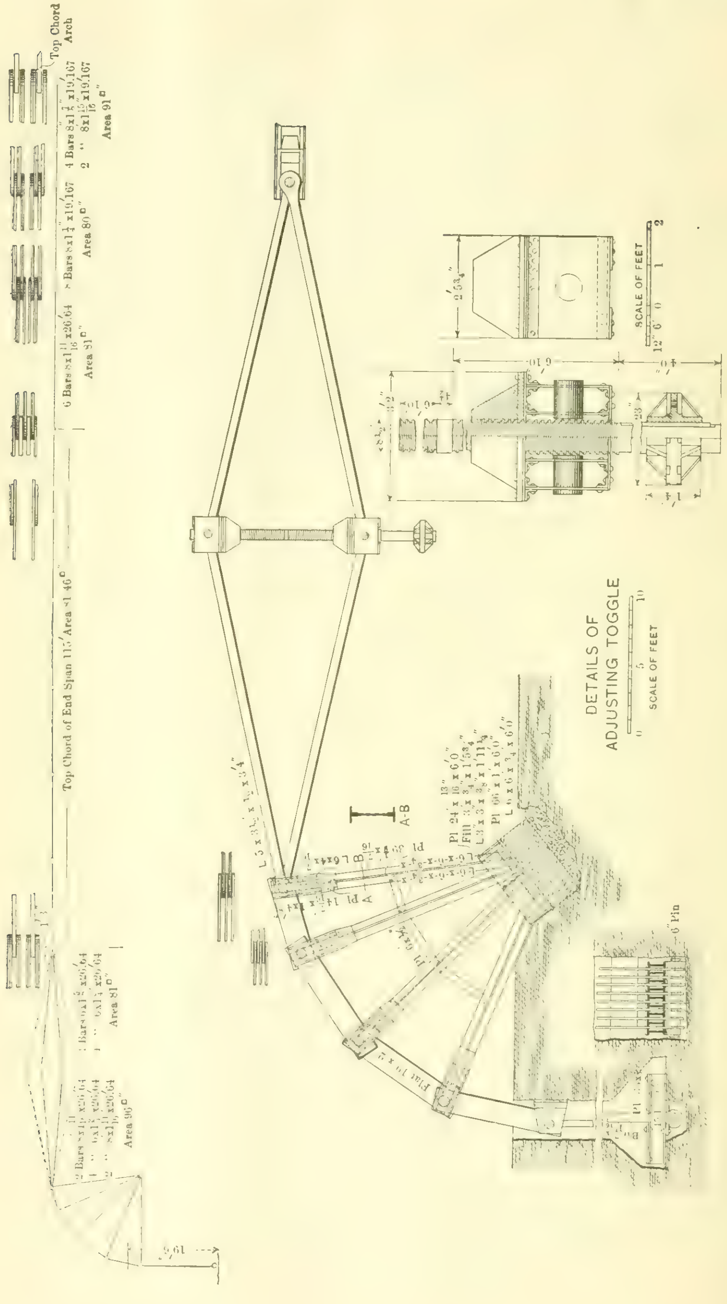

Top Chord Arch
Area 91□"
Area 80□"
Area 91□"
Top Chord of End Span 115' Area 146□
Area 81□"
Area 96□"
A-B
DETAILS OF ADJUSTING TOGGLE
SCALE OF FEET
SCALE OF FEET
6" Pin

FIG. 10.

ported on the towers of the old bridge. This method was used with very satisfactory results by Mr. Buck in the erection of the first Verrugas Viaduct in Peru, but owing to the large cost of a plant suitable for handling heavy and unwieldy pieces and their limited experience in using it for such purposes, it was proposed by the contractors to use travelers resting on the top chords of the arch, and the change was sanctioned by the Chief Engineer. The anchorages were strengthened to accommodate the additional weight. This erecting plant proved very safe and efficient. The two sides were entirely independent of each other, furnishing two points of progress, and when there was no outside cause of delay, the erection proceeded rapidly. There were two engines to each traveler, placed in the towers of the old bridge at the level of the railway floor, this being a good point of observation, and well out of the way.

Travelers.—The metal travelers required considerable special treatment, to clear the cables, and furnish the necessary clearance for trains. They are shown on Plates XI, XII and XIV. The heaviest piece handled on this work weighed 32 tons, but the capacity of the travelers was considerably greater. For handling the rib members, special clamps were used to make them lie at the angle of the batter.

Progress of Work.—The false-work for the end spans was first erected, and the travelers raised on the outer bents, in which position they handled the skewback castings and first panels of the arch. The first sections of the rib rested on light false-work until the end posts and the braces in the first panel were placed. This much of the first panel was then lifted and held clear of the false-work, by ordinary tackle attached to the tops of the end posts. The false-work is shown in Fig. 1, Plate XI.

When the first top chord sections and the second pair of posts were placed, the pins were driven at the top of the end posts connecting with the anchorages. The end posts were then given the right inclination by means of the adjusting toggle. The traveler was then moved forward on the first panel of the arch, and in this position was ready for the erection of the second panel.

The material was conveyed to the travelers by means of trucks running on tracks on each side of the bridge. These tracks rested on the false-work as far out as the end posts of the arch span, and from

there to the center on the sidewalk brackets. The track stringers for the railway floor were used to carry these temporary tracks, being placed at their proper panels, ready for raising to final position when the railway floor should be put in.

The erection proceeded in this manner to the center. The lower floor system was put in, along with the trusses and lateral bracing. It was dropped below its normal position sufficiently to avoid the possibility of the weight of the old bridge coming on it, when deflected under passing trains, and thereby putting undue stress on the anchorages.

The closure at the middle was anticipated with considerable interest and some anxiety. The absence of the center hinge rendered great accuracy in laying out the work necessary, in order to secure proper closure and distribution of load between the top chord and the rib.

As a safety provision, the center panel top chord sections were not planed to length until six panels of the arch had been completed on each side, and a check measurement had been taken across the intervening space of about 134.5 ft. This measurement was not very satisfactory on account of the difficulties in the way of securing accurate results. The half spans were leaning back from their normal positions, their set and deflection could not be accurately accounted for, and the weather conditions were generally unfavorable. However, it was decided, after taking the measurement, to plane the center chord sections to theoretical length. When the center panels were erected, there remained an opening at the center of 8 ins. due to the two halves of the arch being drawn back, to secure the necessary clearance for placing these panels.

When all was ready, the adjusting toggles were slackened away together. In the proper order of events the top chords should have met first, and then, as those passed from tension to compression, the ribs should have met. But the reverse was the case, the ribs met first, and when the anchorages were entirely slackened off, there was an opening at the center of the top chord of ½ in. This indicated no compression in the top chords at the center, whereas there should have been about 350 tons. The cause or causes of the failure to close, were not, at the time, very obvious, but it was decided that the adjustment could be duly effected after casting off the anchorages. The

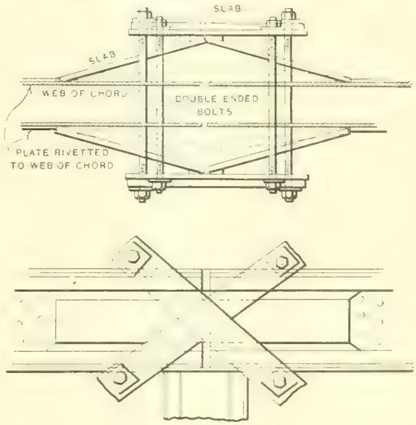
FIG. 1.

FIG. 2.

anchorages were cast off and taken apart. None of the joints were riveted up at this time, but almost all holes were filled with drift pins and bolts. Certain of the rib joints were open, the bearing faces held apart by the drifts and bolts. When these were removed so as to allow the bearing faces to come together, the opening in the top chord at the center was reduced to $\frac{1}{4}$ in.

It then became necessary, in order to secure the required compression in the top chord, to force it apart at the center and insert a shim. This was done by means of a compression toggle, shown roughly in Fig. 11. This toggle was improvised largely from material on the ground. The chords were forced apart until the opening was 1 in. wide, and a shim conforming to the section of the chord and of this thickness was inserted.

Both before and after the top chords were forced apart at the center, levels were carefully taken at each panel point for the purpose of obtaining the exact camber. The results indicated a slight elevation of the camber over the whole span after the adjustment, and in a closer conformity to the theoretical camber.



SKETCH OF
COMPRESSION TOGGLE USED
TO FORCE APART TOP CHORD
OF ARCH AT CENTER.
FIG. 11.

After the adjustment was effected, and the end spans completed, the lower floor system was raised to its final position. Timbers were laid crosswise on top of the roadway stringers, and when all was ready the stiffening truss was blocked up for its whole length on the new work. This was done between trains. The suspenders were then detached from the cables, and the cables were taken down. The wrapping was cut from the cables with axes, and the strands were cut at the shoes and lowered down, one at a time, on the bridge, where they were cut up for scrap.

After the removal of the cables, the upper floor was put in. In order to do this the upper floor and top chords of the old bridge had

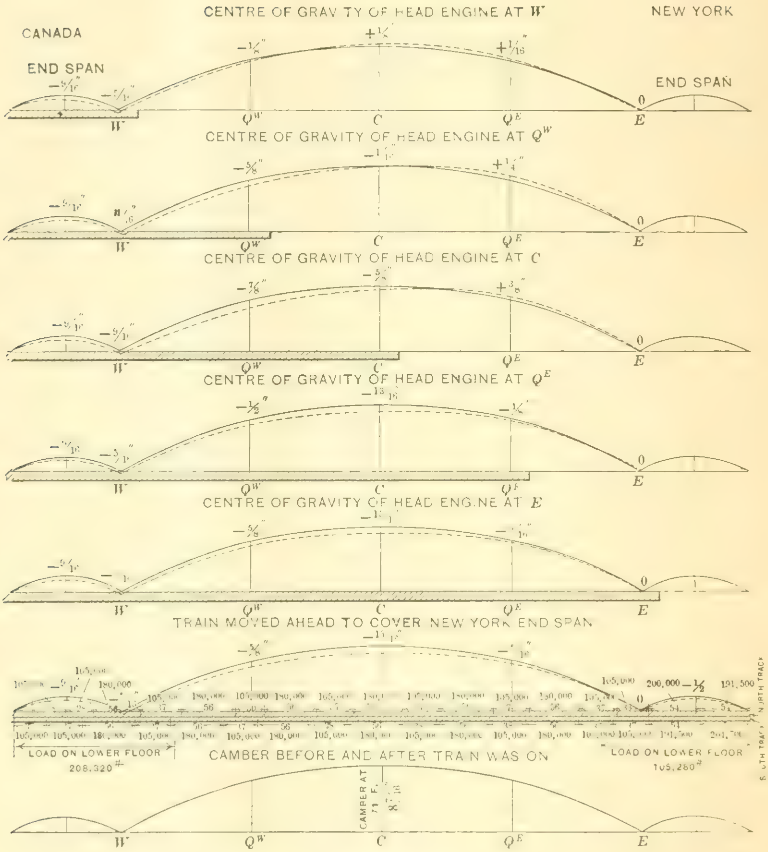


DIAGRAM OF

DEFLECTION OF NIAGARA RAILWAY ARCH

UNDER TEST LOAD OF 2,300 TONS, GR.

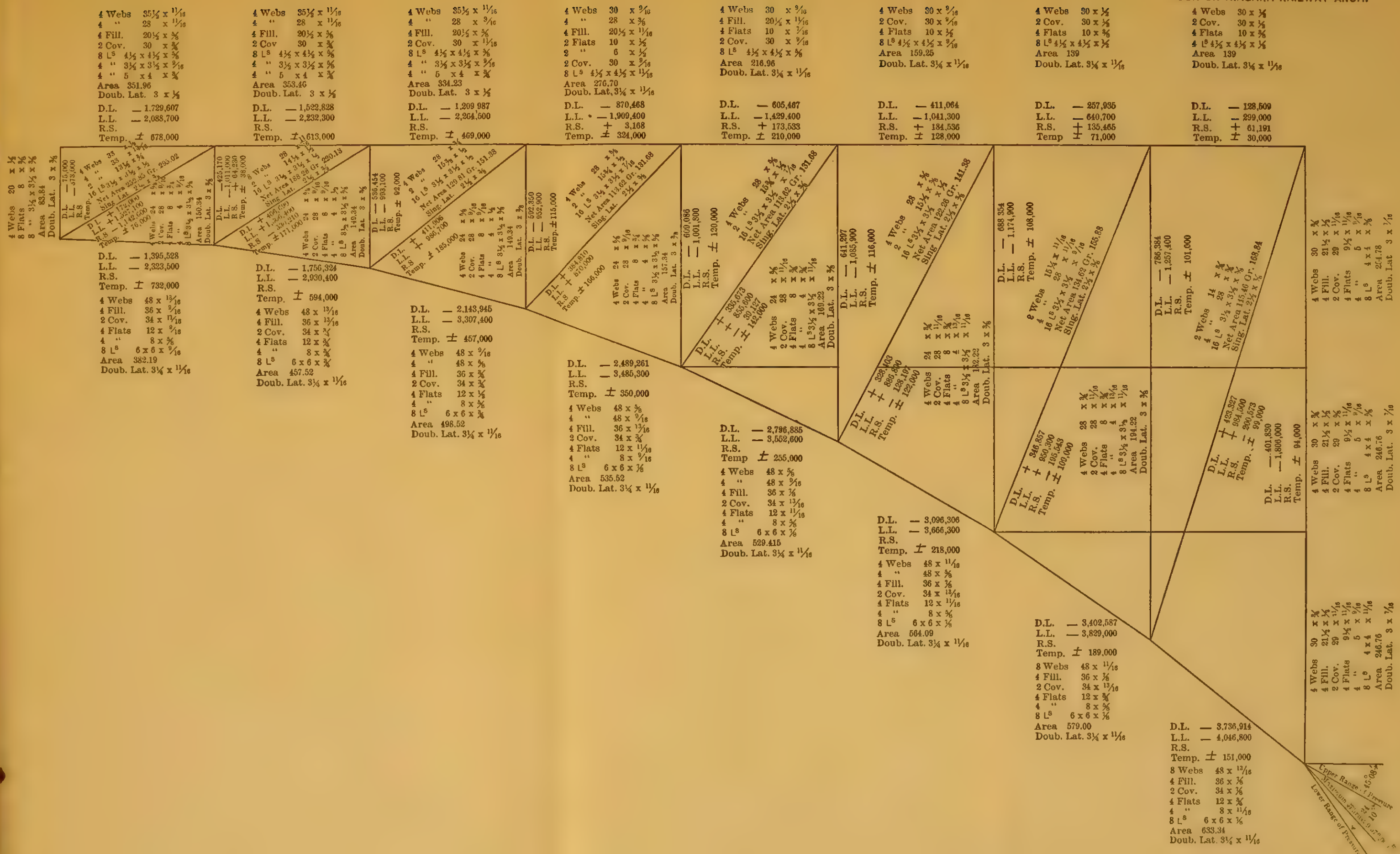
JULY 29, 1897.

FULL LINES REPRESENT THE LINE OF CAMBER OF THE UNLOADED STRUCTURE.

DOTTED LINES REPRESENT DEFLECTED LINE OF CAMBER. THE LINES OF CURVE REPRESENT THE MEANS OF THE OBSERVATIONS ON THE TWO TRUSSES. THESE WERE SO CLOSE THAT THE DIFFERENCES WERE NOT WITHIN THE RANGE OF ACCURACY OF OBSERVATION OR PLOTTING.

FIG. 12.

PLATE XVI.
PAPERS AM. SOC. C. E.
APRIL, 1898.
BUCK ON NIAGARA RAILWAY ARCH.



to be removed. In order not to stop traffic this had to be done between trains, two panels at a time. The top chords and track stringers of the stiffening truss were cut into sections, conforming as closely as possible to the panel lengths of the new bridge, and the panels of the new bridge were put in as the sections of the old bridge were taken out.

Operations began at the middle, and after the first day, when only one panel was placed, two panels a day were put in until all were in place. The time allowed for this work was about two hours each day, and the work was always done within the time limit. The same track alignment was preserved, and the same rails and ties were used temporarily after the new floor beams and stringers were in place. When this work was completed as far back as the shore ends of the end spans, the towers were taken down, a high gin pole being used to remove the caps and upper sections, and the traveler to remove the lower sections.

LIVE LOAD
DIAGRAM OF LOAD ON EACH TRACK

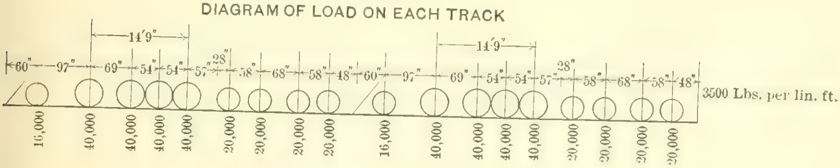


FIG. 13.

After the removal of the towers, the new plate girder approach spans were substituted for the old ones. The maintenance of traffic here, while shifting, was more difficult than on the main and end spans, on account of the switches to be taken care of; but the change was accomplished without mishap. Plates XI to XV show various views of the bridge during construction.

Test.—On account of the difficulty of securing the full load of 10 000 lbs. per running foot, it was decided to make up two test trains as heavy as were available, and to observe the deflections under this loading. Each train consisted of two heavy Lehigh pushers, four of the heaviest Grand Trunk locomotives at hand and nine coal cars. The cars were of 30 tons capacity, loaded with coal, and had as many rails piled on top as was deemed safe for the cars. The loading is given in detail in Fig. 13. As indicated, some load was put on the lower floor, chiefly on the end spans.

The deflections under the test load are shown on Fig. 12.

The apparent slight irregularities in deflection are probably due more to inaccuracies of observation on account of the humid atmosphere and consequent refraction, than to any real irregularity of settlement of the structure under the load. The arch assumed exactly the same camber after the removal of the load as it had before the load was put on.

Ground was broken for the foundations of the arch span April 9th, 1896, and these were completed September 28th, 1896. The contract for the superstructure was let June 15th, 1896. The work of erection began September 17th, 1896, and the bridge was ready to test and was tested July 29th, 1897. All work on the bridge was completed August 27th, 1897.

In conclusion it should be stated that much credit is due to the contractor for the superstructure, the Pennsylvania Steel Company, for its care and efficiency in the prosecution of the work.

1898. APRIL. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

ORIGIN OF THE GULF STREAM AND CIRCULATION OF WATERS IN THE GULF OF MEXICO, WITH SPECIAL REFERENCE TO THE EFFECT ON JETTY CONSTRUCTION.

By N. B. SWEITZER, Jr., Jun. Am. Soc. C. E.

TO BE PRESENTED JUNE 15TH, 1898.

The origin of the Gulf Stream and the circulation in the Gulf of Mexico have never been satisfactorily settled. Many theories have been advanced from time to time, but none as yet have been established upon acceptable proof. The commonly received theory of the present day, that these currents flow in from the South Atlantic, pass the north shore of Yucatan, follow the coast line of Mexico and Texas in a north-northeasterly direction, and finally escape through the Straits of Florida, appears to be plainly contradicted by evidence gathered from the various surveys of the past, together with recent discoveries in connection with the several deep-water projects along the coast of Texas.

The true solution of the question seems to be that the currents coming in through the Straits of Yucatan follow one of two courses,

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

dependent upon the variations of barometric and planetary conditions, viz.:

(1) During the high declination of the moon, coincident with a continued low barometer off Cape Hatteras and a high barometer in the Gulf of Mexico, they flow from the Yucatan Channel in a northeasterly direction around the extreme west coast of Cuba and pass out into the Atlantic through the Straits of Florida, where they become known as the Gulf Stream.

(2) Under opposite atmospheric conditions, and during the low declination of the moon, the Channel of Yucatan pours its waters into the Gulf, so that they spread out in all directions, moving on its center; thence, being deflected by the outward flow of contrary currents, they also pass through the Straits of Florida as the Gulf Stream.

In order to understand these currents aright, especially those in the western part of the Gulf, it will be necessary to review, as briefly as possible, past observations and discoveries concerning the Gulf Stream and equatorial currents. The discovery of America may be said to be due primarily to the Gulf Stream. Long before the time of Columbus, there had been noticed in the débris constantly washed upon the shores of Scotland, Norway and France, pieces of carved wood, and canoes of curious design, quite foreign to the people of the known world. Little was needed, therefore, to convince a reflective mind, like that of the great explorer, that these things came from lands beyond the seas, which could be reached by sailing westward.

As far as known, Columbus was the first navigator to observe the oceanic currents; having noticed, when sounding in the Sargasso Sea, that the lead appeared to recede from the ship, which he rightly interpreted as meaning that the ship drifted away from the lead. Speaking of the strong currents which, on his later voyages, he noticed in the Caribbean Sea, as also among the Antilles and off the coast of Honduras, he says:

"When I left the Dragon's Mouth I found the sea ran so strongly to the westward that between the hour of Mass, when I weighed anchor, and the hour of Complines, I made sixty-five leagues of 4 miles each with gentle winds." He further adds: "I hold it for certain that the waters of the sea move from east to west, with the sky, and that in passing this track they hold a more rapid course and have thus carried away larger tracks of land, and that from hence have resulted the great number of islands."

It was about this time that Sebastian Cabot noted the Labrador current. However, the first authentic mention of the Gulf Stream is found in the log of Antonio De Alaminos, the pilot of Ponce de Leon in his famous expedition for the Fountain of Life in 1513. Having set out from Porto Rico he crossed the stream in the neighborhood of Cape Canaveral, and, after reaching latitude 30° north, he turned and coasted as far as Tortugas, stemming the current for several hundred miles. The log says that they met currents which they were unable to stem even when they had good winds; and though they appeared to be going through the water at a high rate of speed they recognized that they were really drifting backward, and that the stream was stronger than the wind. It states further that when these vessels came to anchor near the coast, one of them, a brig, being in water too deep to anchor, was soon carried out of sight by the stream.

This knowledge subsequently proved of great value to Alaminos when Cortez, in his expedition for the conquest of Mexico, placed him in command of the entire fleet. Later, when it became necessary for Cortez to send envoys to Spain to save his life, he gave Alaminos the speediest vessel of the fleet, with the instructions to sail to the north of Cuba and into the Atlantic through the Florida Straits, thus utilizing the Gulf Stream. This route subsequently became the chief course of navigation between the West Indies and Europe, and played an important part in the later development of the City of Havana.

Sir Humphrey Gilbert in some of his writings seeks to trace the motion of the waters from the African coast to America, and writes:

"The current runs all along the eastern coast of that country northward as far as Cape Freddo, being the farthest known place of the same continent towards the north," from which he concludes that "it must either flow around the north of America into the South Sea, or it must needs strike over upon the coast of Iceland, Norway and Finland."

He accepted the former theory because he was anxious to prove the existence of the northwest passage.

One of the earliest theories advanced to account for the Gulf Stream appears in "La Cosmographie," in which it is claimed that the currents in the Straits of Florida are caused by the rivers emptying into the Gulf of Mexico. John White, Governor of Roanoke, referring to a voyage from Florida Keys to Virginia about 1590, says:

"We lost sight of the coast and stood to sea for to gain the help of the currents, which runneth much swifter farre off than in sight of the coast, for from the Cape of Florida all along the shore are none but eddie currents setting to the south and southwest."

Thus it may be seen: that at this early date the existence of a swift, powerful current was known; that it extended from Florida Keys, "out of sight of the coast," northward beyond Virginia; and, that between this current and the coast were "eddie currents setting to the south and southwest." In other words, the axis of the Gulf Stream had been located and the contrary current flowing southward adjacent to the coast noted.

Isaac Vassius writes, about the year 1663:

"With the general equatorial current the waters run towards Brazil along Guyana and enter the Gulf of Mexico, and from thence turning obliquely, they pass rapidly through the Straits of Bahama. On the one side they bathe the coast of Florida and Virginia and the entire shore of North America, and on the other they run directly east until they reach opposite shores of Europe and Africa."

Here it is seen that the writer notices the entrance of the currents into the Gulf of Mexico, from the direction of the Guyana coast, through the Caribbean Sea, coming into the Gulf between Cuba and Yucatan.

On their entrance into the Gulf he points out that they increase in rapidity, "turning obliquely" towards the Straits of Florida or Bahama. Strange as it may seem, these observations of Vassius, made as long ago as 1663, are much nearer the truth than those of writers as late even as to-day.

It is interesting to compare this information with the article in the "Encyclopædia Britannica" on the Atlantic Ocean (see Fig. 1). Here it is stated that the current

"passes westwards along the northern coast of South America until it is deflected northwards by the coast line of Central America, and is driven between the peninsula of Yucatan and the western extremity of Cuba into the Gulf of Mexico, at the rate of from 30 to 60 miles per day. A portion of it passes direct to the northeast along the northern shore of Cuba; but by far the larger part sweeps round the Gulf, following the course of its coast line, and approaches the coast of Cuba from the northwest as a broad deep stream of no great velocity, seldom running more than 30 miles per day."

In the light of recent investigation, the theory of Vassius appears to have been more nearly right. From the time of Vassius very little

notice seems to have been taken of the Gulf Stream until about the year 1770 when some American whalers, sailing from the Bahama banks, southeast to the Azores and north to Baffin's Bay, made the discovery that the whales stayed north or south of a certain line which afterwards proved to be the Gulf Stream.

This knowledge was communicated to American navigators who found it of great value in their voyages to Europe. About this time Benjamin Franklin took up the matter and published a map which was rejected by the British Government and the English ship captains, who, true to the inherited conservatism of their nature, continued their old course, often arriving at New York when American vessels, which had left England at the same time, were half way across the Atlantic on their return voyage. Upon the outbreak of the war with England, Dr. Franklin suppressed this map, but the knowledge gained from the experience of the whalers proved to be of the greatest possible advantage to the infant navy of the American Colonies.

FIG. 1.

Dr. Franklin now made a number of voyages across the Atlantic, and from this time dates the first truly systematic and scientific investigation of the Gulf Stream. After noting several observations of the temperature made with the aid of the thermometer, Franklin says:

"I find that it (the Gulf Stream) is always warmer than the sea on each side of it, and it will appear from thence that the thermometer may be a useful instrument to the navigator; since currents coming from the northern into the southern seas will probably be found colder than the water of those seas, as the currents from the southern seas are apt to be warmer."

A. von Humboldt next published a remarkable work on the Gulf Stream, in which he maintained that it was not the same at all seasons, but depended to a great extent upon the wind. This is found to be true, at least as far as velocity is concerned.

The excerpt on page 294 is taken from an exhaustive report by Lieutenant Pillsbury, U. S. N.:

"During the first quarter of this century the British Admiralty office had collected a great quantity of material on the subject of ocean currents and meteorology, most of which had never been known to the public. Mr. James Rennell, who had devoted his life to the subject of geography and particularly to ocean currents, was given the task of compiling and collecting data. He combined the results on large charts of the ocean, which were the admiration of the day. He also wrote a volume entitled, 'An Investigation of the Subjects of the Currents in the Atlantic Ocean.'

"Major Rennell adopted Dr. Franklin's theory as to the principal cause of ocean currents, and divided them into two classes, drift currents, caused by constant or long-continued winds on the surface of the water, and stream currents which are formed by the accumulation of water by the drift currents meeting an obstacle and thrown sideways or out of its usual course. The Gulf Steam he placed in the latter class. He considered the water in the North Atlantic a drift current impelled by the prevailing westerly winds, and these also were the cause of the African current."

From these and other observations Major Rennell draws the following conclusions:

(1) "That there existed a change in the position of the column of warm water from time to time"; (2) "that the breadth varies at times in the proportion of more than two to one"; (3) "that these changes had been observed some time to be very sudden, *e. g.*, on one occasion the stream had been found to be 140 miles in width, and in two weeks later at the same spot it was 320 miles;" (4) "that these changes did not follow any regular course of the seasons, for it was 320 miles wide in May, 1820, and only 186 miles in May, 1821, nearly at the same place;" (5) "that on the northern side of the stream the body of warm water is more permanent than on the south, and that the warmest water is found to the north, as if indicating the strongest part of the stream there;" (6) "that the existence of warm water does not necessarily indicate the presence of the stream, but must be regarded as an overflow or deposit of superabundant water; or even from a counter current;" (7) "that there are without doubt veins of colder water within the body of warm currents."

These deductions of Major Rennell were a vast advance upon anything that had previously been attempted. True, some of them are faulty, but it must be remembered that the temperature observations were those of the surface, and that surface-water, impelled by a gale of wind, will traverse many hundreds of miles in a short space of time. Besides this, his conclusions were gathered from totally unconnected data, such as reports of merchantmen, naval officers, and ship captains

of all classes, whose observations, especially for longitude, were often careless in method and detail. A period of about twenty years elapsed before anything further was accomplished in the exploration of the Gulf Stream. Then, in the year 1844, Professor A. W. Bache, of the United States Coast Survey, began the first really scientific and comprehensive survey of this great ocean river.

Origin of Ocean Currents.—It will be necessary to notice some of the more important theories advanced. Columbus held the opinion that the waters followed the motions of the heavens about the earth, in which view Sir Humphrey Gilbert concurred. Kepler believed the flow of the currents to be considerably influenced by the motion of the earth; the cohesion of the particles of water being less than that of land, the water was naturally left behind in the revolution of the earth on its axis.

The scientific world at that time was considerably agitated by the opposing theories of Passine and Furnier. The former held that the currents were caused by the heat of the sun attracting the ocean, forming an immense mountain of water, which vessels had great difficulty in ascending. He thought that this mountain moved constantly westward until it met the South American coast, which turned it and caused it to move northwestward. Furnier, on the contrary, claimed that the sun caused an immense hole to be made by evaporation into which the waters rushed, thus causing the currents. Like the fable of the two knights on opposite sides of the statue, they were both right to a certain extent.

Kircher, a learned mathematician of Würzburg, attributed the currents to the effect of the winds, which, when deflected by the shore line, formed currents now designated as stream currents. He also noticed the influence of the moon on these currents.

Dr. Franklin strongly advocated the theory that the constant winds blowing towards South America caused a rising in the level of the water, which, being deflected mostly northward, entered the "Bay of Mexico," flowing north of Cuba, and from thence to the Banks of Newfoundland. Von Humboldt attributed the Gulf Stream to the winds, the melting of ice in the polar regions, and the revolution of the earth on its axis.

Lieutenant Maury, U. S. N., in his valuable work entitled "Physical Geography of the Sea," says that the ocean currents are due to

various causes, the more important of which are: *first*, the difference in the specific gravity of sea water in the tropics and in the polar regions; *second*, the influence of the winds. He says:

"Difference of specific gravity is the *chief* cause. Whenever the waters in one part of the sea differ in specific gravity from the waters in another part, no matter from what cause the difference may arise, or how great may be the distance between two such parts of the sea, the heavier water will flow (by the shortest route) towards the lighter; and the lighter in its turn will seek the place from whence the heavier came."

"In other words, from whatever part of the sea a current flows, back to that part a current of equal volume must flow."

He then introduces two qualifying theories to explain the western flow of the equatorial current, viz., the rotation of the earth and the winds. There is no doubt that the evaporation in mid-Atlantic, in the latitude of Cape St. Roque, between the continents of Africa and South America, far exceeds the precipitation, and consequently the specific gravity there must be greater than that of either the Arctic or Antarctic Oceans, which are little more than brackish.

If, therefore, Lieut. Maury is correct in his supposition, that the difference of specific gravity is the chief cause of oceanic circulation, then there would necessarily be a flow of the heavier saline waters from the equator direct to the south and north, and a consequent return of the lighter and fresher waters from the polar regions.

Such, however, is not the case, and to account for the flow of ocean currents another cause than that of specific gravity must be sought. This cause will be found in the winds which blow constantly from Africa to South America and the West Indies. The equatorial current moves directly west, unaffected by its specific gravity. The rotation of the earth doubtless does affect the ocean currents, but its effect is far less than that of the winds. Specific gravity too may be a considerable aid to oceanic circulation, but it is a mistake to suppose it to be the prime cause. Difference of barometric pressure, especially in shallow bays and gulfs, perceptibly affects the currents by accelerating or retarding their velocity. The effect of this force is seen to be very great when it is remembered that a difference of 1 in. in the barometer will be accompanied by a difference in the elevation of the Gulf of more than a foot. Taking into consideration the vast quantity of water involved in such elevation, one can realize something of the tremendous volume which pours into the Atlantic.

Circulation in the Gulf of Mexico.—Having reviewed the history of the Gulf Stream investigation and the more important of the theories advanced to account for its origin, the original question, concerning the course followed by the currents between the time of their entrance into the Gulf of Mexico at Yucatan, and their departure through the Straits of Florida, will be considered.

The question has never been regarded as satisfactorily answered, and it is a remarkable fact that even eminent authorities like Maury and Pillsbury seem to evade it. Maury says :

"It (the current) enters the Caribbean Sea and the Gulf of Mexico, from whence it issues through the Straits of Florida as the well-known Gulf Stream."

Lieutenant Pillsbury says in his report, page 591 :

"We have thus followed the water driven by the *vis a tergo* of the trade winds from the coast of Africa to the Yucatan Channel, from which it flows into the Gulf of Mexico and through the Straits of Florida into the Atlantic."

Reference has already been made to the popular opinion, as expressed in the "Encyclopædia Britannica," viz., that the waters come in through the Yucatan Channel and follow the coast line of Mexico and Texas, finally departing through the Florida Straits; but a more reasonable suggestion is found in the report of the Superintendent of the United States Coast Survey published in 1895, in which he states that the waters of the Gulf of Mexico are "erratic in direction, and feeble in force," and suggests what appears to be the correct theory, that the directions of the currents entering and leaving the Gulf are dependent upon the declination of the moon and certain conditions of barometric pressure.

The winds can have but little influence over the waters in passing through these channels, as they are deep-streamed currents, flowing at a probable depth of from 1 000 to 1 500 fathoms; but the circulation in the Gulf is unmistakedly dependent to a large extent upon the winds.

It was once thought by many that the Mississippi River was the "fountain-head of the Gulf Stream" and caused a circular motion in the Gulf by flowing direct from its mouth to the Straits of Florida. The absurdity of this idea is readily seen from the fact that the amount of water emptied into the Gulf by this river is only 0.36 cubic mile per twenty-four hours,* while the amount received through the

* Humphreys and Abbot.

Yucatan Channel is 652 cubic miles in the same length of time. This fancy is still further disposed of by the discovery that the current of the Mississippi enters the western part of the Gulf and flows in a southwesterly direction.

Here is a good illustration of the opportunity afforded of tracing a stream of fresh water into the sea by means of the difference of density; for by this means the waters of the Mississippi are easily traceable from its mouth in a southwesterly direction, to a point only a little south of the latitude of the Rio Grande, and not far east of the longitude of Galveston.

The specific gravity theory will now be considered. The specific gravity map of the Gulf* shows that the line of maximum density extends from about 300 miles north of Yucatan nearly west to the eastern shores of Mexico. The northern half of the Gulf is quite light, except off the western coast of Florida. If, then, this specific gravity theory be the right one, there will be a flow from the east, southeast and south towards the north and northwest. This, to a certain extent, is true, especially where the motion of the waters is accelerated by the prevailing winds which produce drift currents along the northern and western shores of the Gulf. For nearly ten months in the year these winds blow constantly from the south to a little north of east and for at least nine months they seldom ever change from the southeast. During these months it is not only constant, but strong, and causes the waters to move towards the northwest. When they reach the northern part of the Gulf they are deflected by the shore line and move westward. On the other hand the waters from the southwestern part impinge on the coast of Mexico, and produce the current which flows north along the shores of Mexico and Texas.

These two currents meet in the western part of the Gulf, raising the surface and thereby producing a return current which passes south and eastward towards the Straits of Florida. The current along the coast of Mexico is feeble in force near its starting point, off the west of the Yucatan peninsula, but gradually increases until it reaches its maximum velocity between Tampico and the Rio Grande, where it flows at the rate of 3 miles an hour, and then continues to diminish until it meets the contrary current from the east.

The circulation of the waters in the Gulf is shown in Fig. 2.

* Report of the United States Coast Survey for 1895, p. 369.

A considerable agitation of the waters, covering an area of about 100 square miles, occurs off the west coast of Texas, about 40 miles south and 20 miles east of Aransas Pass, which can only be accounted for as resulting from the meeting of two opposing currents. In the immediate neighborhood of this phenomenon the coast is covered with débris of every description, among which the fir tree of Wisconsin, the palmetto of Florida, the cocoanut and other products peculiar to the tropical regions of the South are found lying side by side.

The chief obstacle to harbor improvements along the coast of Texas is the vast quantity of sand which, being stirred up and held in suspension by the waves, is carried by the currents and deposited in the channels of harbors. A current, to carry heavy sands, must have a bottom velocity of about 2 ft. per second, thus necessitating a much greater surface velocity, which it is known these currents do not have, of themselves, under ordinary circumstances, as they rarely move faster than $2\frac{1}{2}$ miles per hour. Therefore, it is concluded that the wave action must be the chief factor in the dislodgement of the sands, and, in connection with the littoral currents, determines the position and form of harbor jetties.

FIG. 2.

The author's attention was first attracted to the littoral currents by the construction of a north jetty at Aransas Pass, Texas, in 1895. Prior to that time attempts had been made to obtain deep water by the construction of jetties on the south side of the channel, but they failed through the lack of capital required for their completion.

In that year the Aransas Harbor Company built a jetty in the form of a letter S, on the north side of the channel, running east and west half a mile from the southeast extremity of St. Joseph's Island. This work was conducted upon the supposition that the littoral current came from the north, and that it would be so deflected by the jetty as to enter Aransas Bay between the western end of the jetty and St.

Joseph's Island. It was further imagined that during ebb tide this current would meet the outgoing current in the Pass, thus neutralizing both velocities and preventing the outward flow from passing north of the jetty; or, if the ebb current was the stronger, they would pass out through the channel on the south side of the jetty.

Long before this work was completed the author became satisfied that the littoral current came from the south, as, by a series of experiments, he had previously proved to be the case at Pass Caballo, not far north of Aransas Pass, where the conditions were the same.

When the jetty was completed it was found that there was less water in the channel than before the work began. Colonel Goodyear, the last contractor, recognized the mistake that had been made, and proceeded to finish the jetty on the south side of the Pass, thus excluding the drift sand brought in by the currents and waves from the south, with the gratifying result that the author's latest survey, made in June, 1897, showed a marked increase in the depth of the channel. Further evidence is unnecessary to establish the fact that, at this point at least, the littoral currents flow from the south.

If the jetty construction upon the coasts of Texas and Mexico is examined, it will be found that these currents maintain a uniformly northern course from Tampico to Galveston, where they meet those coming from the westward from the mouth of the Mississippi.

At Tampico the south jetty was built first in order to protect the channel from southern sand-bearing currents. At Brazos, Santiago, Texas, the same thing was done. Lieutenant G. A. Zimm,* writes:

"The direction of the channel across the bar depends upon the direction of the winds and littoral currents. During nine months of the year southerly winds blow and there is a littoral current from the south."

The same thing is noticed on the sketch accompanying Captain McClellan's report on this harbor in 1853. At Pass Cabal only one jetty was built, and that was also placed on the south side. A jetty was commenced at the mouth of the Brazos River on the north side, but Major Ernst, in his report of September 6th, 1887, remarks:

"The map of recent survey shows the channel, instead of running southeast in the direction intended, turned off at a right angle and running northeast across the jetty."

* Annual Report of Chief of Engineers, U. S. A., page 1330.

At Galveston the South Jetty was built first. At passes east of Galveston, however, the jetties are almost invariably constructed on the east side, to check the sand-bearing currents in their westward flow. Among these may be mentioned Calcasieu River Pass, La.;* also Sabine River Pass.†

It is well known that at the mouth of the Mississippi the littoral currents flow from the east. Hence, it may be concluded that the currents flowing westward are on the northern shore of the Gulf, and those flowing north along the western shore meet at some point near Galveston, the location of which is variable, as it is dependent upon the direction and force of the winds.

Summing up these observations of the Gulf currents, it is found that there is abundant evidence of the presence of two sets of currents in the Gulf of Mexico, viz., deep-stream currents and littoral-drift currents.

The former enter the Gulf through the Yucatan Channel, and, under certain barometric and planetary conditions, pass by the western extremity of Cuba and flow out through the Straits of Florida, or else, under converse planetary and atmospheric conditions, they spread out over the Gulf in all directions, moving on its center.

The littoral-drift currents are originated through the agency of the prevailing southeast winds and flow northward along the western boundary of the Gulf, and west along the northern boundary, meeting in the vicinity of Galveston and forming a stream-current which flows in a southeasterly direction towards the Straits of Florida.

In the past there has been far too little attention paid to the motion of the Gulf waters. Were they better understood, there can be no doubt that vast sums of public and private capital might have been expended more judiciously than they have been, resulting in more real and lasting good to the people of the coast country. If, therefore, this paper is the means of inducing a greater interest in these Gulf currents, which have such a direct and important bearing on trade and commerce, the author will feel that it has successfully accomplished its mission.

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* Report of Major W. H. Heuer, 1886.

† Report of Secretary of War for 1895.

MEMOIRS OF DECEASED MEMBERS.

Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS DOANE, M. Am. Soc. C. E.*

Died October 22d, 1897.

The name of Doane is exceedingly old; it existed as far back as the year 1000. Doanes went over to England from Normandy with William the Conqueror; Doanes were prominent in English Church History, and there were families of that name in and about Chester, England.

The first of the name known in America was Deacon John Doane. This progenitor of all the Doanes in the United States and British America came from Wales in the ship *Fortune*, next and shortly after the *Mayflower*, in 1621, to Plymouth, Mass. He lived there until 1644, when, with Governor Prince and other associates, he sailed from Plymouth across the bay and founded the Town of Eastham. He died in 1686, at the age of 96 years.

Thomas Doane, the subject of this memoir and a direct descendant of Deacon John Doane, was born in Orleans, on Cape Cod, Mass., September 20th, 1821. His parents were John Doane, a native of Orleans, and Polly (Eldridge) Doane, a native of Yarmouthport. His father was a well-known lawyer, serving as a State Senator and otherwise prominent in public life; being the originator of "forest culture" in this country, and a promoter of the culture of fruit trees on the Cape.

Thomas Doane was the eldest of eight children, and his early education was received at an academy established by his father and other well-known citizens of the Cape District who had children to educate. Leaving this school at the age of nineteen, he then attended the English Academy at Andover, Mass., for five terms, after which he entered the office of Samuel M. Felton, one of the noted civil engineers of his time, and a leading citizen of Charlestown (now Boston), Mass. As was the custom for engineering students in those days, Mr. Doane remained for three years with Mr. Felton, and then entered into active professional employment. He was engaged first as engineer of the Windsor White River Division of the Vermont Central Railroad,

* Memoir prepared by F. W. D. Holbrook, M. Am. Soc. C. E., who was, at intervals, for a number of years Mr. Doane's Chief Assistant, from information mainly furnished by public prints, and from personal knowledge.

and from 1847 to 1849, as Resident Engineer of the Cheshire Railroad at Walpole, N. H.

In December, 1849, he returned to Charlestown, Mass., and opened an office, where he carried on a general civil engineering and surveying practice, either personally or (when necessarily absent in the conduct of large enterprises) through capable assistants, until the time of his death.

At one time or another Mr. Doane was connected with all the railroads running out of Boston, and particularly with the Boston and Maine. In 1863, the State of Massachusetts having assumed charge of the completion of the Hoosac Tunnel, on which some little work had already been done by contractors, Mr. Doane was appointed Chief Engineer under a Board of State Commissioners, of which Mr. John W. Brooks was chairman. The work to be done involved a total change in the methods followed up to that time, and the introduction of modern ideas and appliances. As Chief Engineer, Mr. Doane relocated the tunnel line and established its grades; connected the two ends by precise measurements and levels over the mountain, thus ensuring great accuracy in the final meeting of the borings; built the dam across the Deerfield River to furnish water-power for turbines to operate air compressors, and a machine shop; and instituted careful experiments on drill steels and many kinds of fuses and explosives. The successful use of nitro-glycerine, the drilling by machine drills operated by compressed air, and the "simultaneous blasting" by electricity were here established for the first time in this country. In his book on tunneling, Mr. Henry S. Drinker pays a high tribute to Mr. Doane, and says that to his

"Persistent energy, far-seeing sagacity, and his able management, we, in a large measure, and, in fact, chiefly, owe the development and introduction into this country of the present advanced system of tunneling with machinery and high explosives. It was under his direction as Engineer of the Commission that the State Experiments were made, and the long and disheartening fight carried through which terminated in favor of the new system. The system which has since given us the Burleigh, Ingersoll and Wood drills, and which also first showed Americans, practically, what the potent agency of nitro-glycerine, first applied by Nobel in Europe, actually was."

Mr. Doane gave much time and thought to the perfection of compressed air machinery. The machine drills devised and used at the tunnel owed much of their efficiency to him; and the carriages on which they were operated were of his invention. He has been designated the "Pioneer" of compressed air in this country. As early as 1873 he proposed a compressed air power plant to do away with the endless number of boilers and fires that are used in closely built cities, and he printed at that time an article which contained ideas agreeing with many being brought out and advocated at the present time. In this matter he was many years in advance of the day.

In 1869, Mr. Doane went west as Chief Engineer of the Burlington and Missouri River Railroad in Nebraska, an extension of the Chicago, Burlington and Quincy System, and in about four years completed 241 miles of railroad, besides establishing a steam ferry-boat service across the Missouri River at Plattsmouth, and constructing and maintaining a telegraph line the full length of the road.

The names of the towns between Plattsmouth and Kearney were due to him; hence the recurrence of many Massachusetts names, such as Dorchester, Harvard, Lowell, etc.

Mr. Doane made a special study of grades for this railroad line, and the road was built with a view to great economy in operation. Time has proven the soundness of his judgment in the advocacy and construction of the system of low grades he there established. This road, for a prairie country, was exceptionally well constructed in all respects. The leading streams were crossed by Howe truss bridges on masonry abutments; screw-pile piers were used at the two crossings of Salt River; the track was laid on oak ties, and the whole road thoroughly drained from end to end.

Mr. Doane returned to Charlestown, Mass., in 1873, and shortly after was reappointed Consulting Engineer of the Hoosac Tunnel and given charge, not only of the tunnel, which for much of its length required a brick lining, but also of the reconstruction of the Troy and Greenfield Railway. This reconstruction involved several changes in location and a large amount of heavy work in the way of rock and earth excavation, masonry retaining walls, bridge abutments, piers and drainage culverts. Much of the work was of a kind seldom encountered: the road along the bank of the Deerfield River being exposed to heavy wash and mountain slides on the one side, and on the other to damage from the river which was subject to heavy freshets, ice gorges, etc. One noticeable structure on this line was the bridge at Bardwell's Ferry, consisting of several spans of iron truss bridging on masonry supports. The piers of this bridge differed in form from those usually adopted, being of elliptical shape. A description of the bridge may be found in the *Railroad Gazette* of that time.

On February 9th, 1875, at the opening of the tunnel, Mr. Doane ran the first locomotive through it, and he remained in charge of construction until 1877.

Two years later, in 1879, he was appointed Consulting and Acting Chief Engineer of the Northern Pacific Railroad for one year. During this time he located the Pend d'Oreille Division, across the Columbia Plains, in the Territory of Washington, and part of the Missouri Division in Dakota. He constructed and operated a bridge on the ice of the Missouri River between Bismarck and Mandan, in order to save delay in the transportation of railway supplies and material. He also made a thorough reorganization of the engineering

force of the road. Since then, for the past few years, Mr. Doane devoted himself mainly to office practice as a consulting engineer, for which he was much in request.

While in Nebraska, Mr. Doane took a leading part in the agitation of the question of establishing a college there, and secured for its site a square mile of ground beautifully located on the "Big Blue," at Crete, 20 miles west of Lincoln. He also made a large financial contribution toward securing other property, and in recognition of his services as its founder, the institution was named "Doane College." For many years he has been one of its trustees. The bulk of his estate is, by his will, to go to the College ultimately as an endowment. Mr. Doane was also one of the founders of the first bank established in Crete.

During the many years that Mr. Doane followed his profession, he received into his office many young men as students, following an old custom which has to a great extent been superseded in this day of technical schools. Among these may be mentioned J. Herbert Shedd, Samuel L. Minot, John L. Emerson, John A. Cole, J. H. Danforth, Gorham P. Low, James Francis, G. M. Thompson and C. A. Pearson. To these names may be added that of the writer of this memoir.

Some of the engineers who have at times served under Mr. Doane with ability, though not students in his office, are : D. H. Ainsworth, author of "Recollections of a Civil Engineer ;" R. B. C. Bement, I. S. P. Weeks, J. W. Kendrick, W. C. Wetherill, F. H. Clement, W. L. Darling and Jules Breuchaud. Nearly all of those mentioned above are to-day Members of the American Society of Civil Engineers, and are actively engaged in professional work.

Mr. Doane was for over twenty years a member of the Boston Society of Civil Engineers, and for nine years its President. He was a Justice of the Peace for over thirty years. He was for forty-five years a member of Winthrop Church in Charlestown, and for fourteen years one of its deacons. He was a Director of the Associated Charities of Boston, and President of the Charlestown Branch of the organization. He was Vice-President of the Hunt Asylum for Destitute Children; was a member of the New England Historic Genealogical Society; of the Congregational Club; the Bunker Hill Boys' Club and the American College and Educational Society. He was the first President of the Charlestown Branch of the Young Men's Christian Association, and contributed liberally to its support.

Mr. Doane was married November 5th, 1850, to Miss Sophia D. Clarke, who died December 1st, 1868. From this union there were five children.

Later in life Mr. Doane married again. His second wife survives him, as do also four children of his first marriage, viz. : Mrs. David B. Perry, wife of the President of Doane College; Mrs. W. O. Weeden,

wife of a Congregational minister; Mrs. H. B. Twombly and the Rev. John Doane, of Plymouth Church, Lincoln, Neb. Mr. Doane also leaves a brother, Captain Charles Doane.

Mr. Doane was a man of high principles and unswerving integrity, kind and considerate to all associated with him, generous with his purse to all worthy objects, and he lived an earnest and Christian life.

As an engineer, his sound judgment, thoroughness, industry, energy, practical attainments and love of accuracy secured success in all enterprises committed to his charge. His loss will be deeply regretted in many directions.

Mr. Doane was elected a Member of the American Society of Civil Engineers, June 7th, 1882. He died away from home, of heart failure, at West Townsend, Vt., where he had gone with Mrs. Doane to visit relatives.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

By Charles Warren Hunt, M. Am. Soc. C. E.

Cloth, 6x9 Inches.

Printed by order of the Board of Direction of the American Society of Civil Engineers, to be sold only on subscription. The proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

This action has been taken in the belief that many members will welcome the opportunity of contributing something to the building fund.

There are a few copies still on hand of the first lot bound and these can be obtained by those who so desire at $10 per copy.

Orders should be sent to the Secretary.

The book begins with a brief statement of the first movement to form a National Society of American Engineers in 1839. The organization of the American Society of Civil Engineers and Architects in 1852 is then described, a list of its promoters and charter members given, and the work accomplished in its first two years of life sketched. The reorganization of the Association in 1867 and the important events in its career from that date to 1873, when the first publication was issued, are then given in chronological order. Succeeding chapters are under the following heads: Locations Occupied by the Society, Library, International Exhibitions, Publications, Badge, Constitutional Changes and Work Accomplished. Under the head of "Comparative Growth of National Engineering Societies" short sketches of the Institution of Civil Engineers and the Société des Ingénieurs Civils are given. The illustrations consist of 35 half-tone portraits of past officers of the Society and one diagram, all handsomely printed on heavy paper.

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

May, 1898

PROCEEDINGS = VOL. XXIV—NO. 5

Published at the House of the Society, 220 West Fifty-seventh Street, New York, the Fourth Wednesday of each Month, except July and August.

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Vol. XXIV. MAY, 1898. No. 5.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publication.

CONTENTS.

SOCIETY AFFAIRS.

PAPERS.

ILLUSTRATIONS.

The prices of publications are as follows: Proceedings, $6 per annum; Transactions, $10 per annum. Postage will be added when they are sent to foreign countries.

American Society of Civil Engineers.

OFFICERS FOR 1898.

President, ALPHONSE FTELEY.

Vice-Presidents.

Term expires January, 1899:	*Term expires January, 1900:*
GEORGE H. MENDELL,	EDWARD P. NORTH,
JOHN F. WALLACE.	FREDERIC P. STEARNS.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOHN THOMSON.

Directors.

Term expires January, 1899:	*Term expires January, 1900:*	*Term expires January, 1901:*
GEORGE A. JUST,	JAMES OWEN,	S. L. F. DEYO,
WM. BARCLAY PARSONS,	RUDOLPH HERING,	JOHN KENNEDY,
HORACE SEE,	HENRY G. MORSE,	HENRY MANLEY,
JOHN R. FREEMAN,	BENJAMIN L. CROSBY,	CHARLES C. SCHNEIDER
DANIEL BONTECOU,	HENRY S. HAINES,	JOHN J. McVEAN,
THOMAS W. SYMONS.	LORENZO M. JOHNSON.	GEORGE Y. WISNER.

Assistant Secretary, T. J. McMINN.

Standing Committees.

THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

On Finance:	*On Publications:*	*On Library:*
HORACE SEE,	EDWARD P. NORTH,	WM. BARCLAY PARSONS,
S. L. F. DEYO,	JOHN THOMSON,	CHARLES WARREN HUNT,
JAMES OWEN,	RUDOLPH HERING,	GEORGE A. JUST,
JOHN R. FREEMAN,	JOHN F. WALLACE,	FREDERIC P. STEARNS,
DANIEL BONTECOU.	HENRY S. HAINES.	JOHN KENNEDY.

Special Committees.

ON UNIFORM STANDARD TIME:—Sandford Fleming, Charles Paine, Theodore N. Ely, J. M. Toucey, T. Egleston.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd, A. E. Hunt.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, O. M. Carter, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open from 9 to 22 o'clock every day, except on Sundays, when the hours are from 14 to 19 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - 2236-38th Street.
CABLE ADDRESS, - - - - "Ceas, New York."

Vol. XXIV. MAY, 1898. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

May 4th, 1898.—The meeting was called to order at 20.30 o'clock, President Alphonse Fteley in the chair; Charles Warren Hunt, Secretary, and present, also, 80 members and 7 visitors.

The minutes of the meetings of April 6th and 20th, 1898, were approved as printed in *Proceedings* for April, 1898.

A paper by Henry N. Ogden, Jun. Am. Soc. C. E., entitled, "Flushing in Pipe Sewers," was presented by the author. Correspondence on the subject from Mr. W. B. Landreth was presented by the Secretary. The paper was discussed orally by Messrs. Rudolph Hering, J. H. Fuertes, G. W. Tillson and H. F. Dunham.

Ballots were canvassed and the following candidates declared elected.

As Members.

William Anderson Aycrigg, New York City.
Fred Keeler Betts, Valhalla, N. Y.
Le Grand Brown, Rochester, N. Y.
Luther Dean, Taunton, Mass.
James Harvey Edwards, East Berlin, Conn.
George Morrall Gadsden, Montgomery, Ala.
Robert Hall Gresham, San Antonio, Tex.
Wellington Barnes Lee, Hillburn, N. Y.
George King McCormick, Knoxville, Tenn.
Alexander Rice McKim, New York City.
John Charles O'Melveny, Salt Lake City, Utah.
Joseph Strachan, Brooklyn, N. Y.
John Charles Temple, Philadelphia, Pa.
David Williams, St. Johnsbury, Vt.

As Associate Members.

Charles Ames Alden, Steelton, Pa.
Frank William Allen, New York City.
Ernest Howard Baldwin, Clinton, Mass.
Charlie Elsworth Chester, Champaign, Ill.
John Henry Cook, Paterson, N. J.
Albert Sears Crane, Brooklyn, N. Y.
Jacob Anthony Harman, Peoria, Ill.
John Cranch Moses, Boston, Mass.
Anton Schneider, Salt Lake City, Utah.
Joseph Emory Sirrine, Cordova, Ala.

The Secretary announced the election by the Board of Direction on May 3d, 1898, of the following candidates:

As Associate.

William Garrigues Hartranft, Philadelphia, Pa.

As Juniors.

Herman Conrow, New York City.
De Forest Halsted Dixon, Ithaca, N. Y.
Julius Kahn, New York City.
William Sutton McFetridge, Greenville, Pa.
Solomon Mark Swaab, Philadelphia, Pa.

The Secretary announced that a canvass was made on May 3d, 1898, by the Board of Direction, of special ballots, as provided for in Art. III, Sec. 5, of the Constitution, and resulted in the election of Charles Evan Fowler, of Youngstown, O., as Member, and of Peyton Brown Winfree, of Bradford, Pa., as Associate Member.

The Secretary announced the death of William Noyes Taintor, elected Junior, September 3d, 1895; died April 8th, 1898.

The Secretary announced that at a meeting of the Board of Direction May 3d, 1898, certain proposed amendments to the Constitution were considered and were recommended by the Board of Direction for adoption by the Society (see Announcements, p. 89).

The President announced the receipt of two invitations, one from the Seventh International Congress on Navigation to be held in Brussels, July 25th to 30th, 1898, and the other from the *Société des Ingenieurs Civils de France*, to the Fiftieth Anniversary of that Society, to be celebrated June 10th to 13th, 1898.

Any member intending to visit Europe during June or July, and desiring to be accredited to either or both of these meetings, should communicate at once with the Secretary.

Adjourned.

May 18th, 1898.—The meeting was called to order at 20.30 o'clock, Joseph M. Knap, M. Am. Soc. C. E., in the chair; Charles Warren Hunt, Secretary, and present, also, 120 members and 44 guests.

A paper by R. S. Buck, M. Am. Soc. C. E., entitled "The Niagara Railway Arch," was presented by the author and illustrated by the stereopticon.

The Secretary presented correspondence on the subject from Mr. Henry Goldmark. The paper was discussed orally by Messrs. J. W. Schaub, M. Lewinson, Gustave Lindenthal, F. W. Skinner, O. F. Nichols, C. E. Emery, L. L. Buck, and the author.

The Secretary announced the death of Charles Edward Newham, elected Member December 7th, 1887; died February 1st, 1898; and Edward Curtis Rice, elected Member April 7th, 1875; died April 21st, 1898.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 3d, 1898.—Past-President Morison in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Hering, Just, Owen, See and Thomson.

The Finance Committee reported, recommending that certain amendments be made to Article VI of the Constitution (see page 89).

The report of the Committee was accepted, and the proposed amendments to Article VI of the Constitution were adopted as the recommendation of the Board of Direction, and the Secretary instructed to report this action to the Society at its next meeting.

Preliminary action was taken toward the formulation of a report to the Society on the matter of the proposed appointment of Special Committees to report on "Rail Joints for Standard Steam Railroads," and on "Paints Used for Structural Work in Engineering."

Two invitations were presented, one from the Seventh International Congress on Navigation to be held in Brussels, July, 1898; and the other from the *Société des Ingenieurs Civils de France*, to send a delegation to the Fiftieth Anniversary of that Society, June 10th to 13th, 1898. The President of the Society was authorized to take such action in these matters as seemed to him proper.

In view of the change of date for holding the Convention of 1898, it was resolved that a number of *Proceedings* be issued for the month of June, 1898, and that no number be issued for August, 1898.

The following resignations were accepted:

T. M. R. Talcott, M. Am. Soc. C. E.
C. B. Marriott, Assoc. Am. Soc. C. E.

The appointment by the President, under authority of the Board, of a Committee of the Board of Direction, and a Local Committee to take charge of the Arrangements for the Annual Convention, was announced.

Ballots were canvassed in the matter of the reconsideration of the ballot on the application of Charles Evan Fowler for membership, and the candidate was declared elected a Member of the Society.

Ballots were canvassed in the matter of the reconsideration of the ballot on the application of Peyton Brown Winfree for membership, and the candidate was declared elected an Associate Member of the Society.

One candidate for Associate and five for Junior were elected (see page 84).

Applications were considered and other routine business transacted. Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society will be open every day hereafter from 9 to 22 o'clock, except on Sundays, when the hours will be from 14 to 19 o'clock.

JUNE NUMBER OF PROCEEDINGS.

Owing to the change in the time of holding the Annual Convention of 1898, the Board of Direction has decided that a Number of *Proceedings* shall be issued for the month of June, 1898, and that no Number be issued for the month of August, 1898.

HISTORICAL SKETCH.

The "*Historical Sketch of the American Society of Civil Engineers*," by Charles Warren Hunt, was printed by order of the Board of Direction, to be sold only on subscription; the proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out.

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the Building Fund.

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This action has been taken in the belief that many members will welcome the opportunity of contributing something to the Building Fund.

There are a few copies still on hand of the first lot bound, and these can be obtained by those who so desire at $10 per copy.

ANNUAL CONVENTION.

The Thirtieth Annual Convention will be held at Detroit, Mich., July 26th–29th, 1898.

The following Committees of Arrangements have been appointed:

Committee of the Board of Direction:

George Y. Wisner.
J. J. McVean. Chas. Warren Hunt.

Local Committee:

George Y. Wisner, *Chairman.*
A. B. Atwater. J. D. Hawks.
C. E. Greene. H. S. Hodge.
D. A. Molitor. J. C. Hutchins.
G. S. Williams.

At as early a date as practicable a circular will be issued, giving information relative to transportation, excursions, hotel rates, etc.

The following papers will be presented:

"Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company at Ogden, Utah," by Charles D. Marx, M. Am. Soc. C. E.; Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, C. E.

"The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel," by E. Herbert Stone, M. Am. Soc. C. E.

"Marine Wood-Borers," by Charles H. Snow, M. Am. Soc. C. E.

"Reservoir System of the Great Lakes of the St. Lawrence Basin: Its Relation to the Problem of Improving the Navigation of These Bodies of Water and Their Connecting Channels," by Captain Hiram M. Chittenden, Corps of Engineers, U. S. A.; With a Mathematical Analysis of the Influence of Reservoirs upon Stream Flow, by James A. Seddon, Esq.

"Dredges and Dredging on the Mississippi River," by J. A. Ockerson, M. Am. Soc. C. E.

"Three-Hinged Masonry Arches; Long Spans Especially Considered," by David A. Molitor, M. Am. Soc. C. E.

The first three papers are published in this number of *Proceedings.* The others will be published in the number for June, 1898.

PROPOSED AMENDMENTS TO THE CONSTITUTION.*

Amend Article VI as follows:

SECTION 3....Strike out the eighth and ninth lines of this section, viz.: "*The accounts and financial books of the Society shall be examined annually by an expert accountant, to be selected by the Board of Direction.*"

SECTION 4 ...In the third line, and after the word "Section," substitute 7 for *8*.

Strike out the words "*Board of Direction*" in line twenty, and substitute therefor the words "*Finance Committee*," and immediately following insert a new clause as follows:

"*He shall have charge of the books of account of the Society, and shall furnish monthly to the Board of Direction a statement of receipts and expenses under their several headings, and also a statement of monthly balances. He shall present annually to the Board of Direction a balance sheet of his books, as of the 31st of December, and shall furnish, from time to time, such other statements as may be required of him.*"

SECTION 6......Strike out this entire Section.

SECTION 7......Change the number of this Section to *6*.

SECTION 8 .....Change the number of this Section to 7.

SECTION 9 .....Strike out the whole of this Section and substitute the following:

"*8.—The Finance Committee shall have immediate supervision of the financial affairs of the Society, shall employ an expert accountant to audit the accounts monthly, shall approve all bills before payment, and shall make recommendations to the Board of Direction as to the investment of moneys, and as to other financial matters.*"

SECTION 10......Change the number of this Section to *9*.

SECTION 11......Change the number of this Section to *10*.

SECTION 12......Change the number of this Section to *11*.

SECTION 13......Change the number of this Section to *12*.

Article VI of the Constitution will then read as follows:

ARTICLE VI.—MANAGEMENT.†

1.—The President shall have a general supervision of the affairs of the Society. He shall preside at meetings of the Society and of the Board of Direction at which he may be present, and shall be *ex-officio* member of all committees. He shall deliver an address at the Annual Convention.

The Vice-Presidents in order of seniority shall preside at meetings in the absence of the President, and discharge his duties in case of a vacancy in the office.

2.—The Board of Direction shall manage the affairs of the Society in conformity to the laws under which the Society is organized and

* See pages 85 and 86.

† Changes made in present Constitution shown in italics.

the provisions of this Constitution. It shall direct the investment and care of the funds of the Society; make appropriations for specific purposes; act upon applications for membership as heretofore provided; take measures to advance the interests of the Society; appoint all its employees; and generally direct its business. The Board of Direction shall make an annual report at the Annual Meeting, transmitting the report of the Treasurer and of other officers, and of Committees.

3.—The Treasurer shall receive all moneys and deposit the same in the name of the Society. He shall invest all funds not needed for current disbursements, as shall be ordered by the Board of Direction. He shall pay all bills, when certified and audited, as provided by this Constitution and by rules to be prescribed by the Board of Direction. He shall make an annual report and such other reports as may be prescribed by the Board of Direction.

The Board of Direction shall secure a satisfactory surety for the faithful performance of his duties by the Treasurer, and shall renew the same during the month of January of each year.

4.—The Secretary shall be a Corporate Member of the Society. He shall be elected annually by the Board of Direction at the meeting to be held within twenty days after the Annual Meeting provided for in Section 7 of Article VI, or at an adjournment thereof, and shall hold the office for one year, or until his successor is elected, provided that a majority of the whole Board of Direction shall be required to elect the Secretary; this vote to be given, if necessary, by letter.

He shall be, under the direction of the President and Board of Direction, the executive officer of the Society.

He will be expected to attend all meetings of the Society and of the Board of Direction; prepare the business therefor, and duly record the proceedings thereof.

He shall see that all moneys due the Society are carefully collected, and without loss transferred to the custody of the Treasurer.

He shall carefully scrutinize all expenditures, and use his best endeavor to secure economy in the administration of the Society.

He shall personally certify the accuracy of all bills or vouchers on which money is to be paid, and shall countersign the checks drawn by the Treasurer against the funds of the Society, when such drafts are known to him to be proper and duly authorized by the *Finance Committee.*

He shall have charge of the books of account of the Society, and shall furnish monthly to the Board of Direction a statement of receipts and expenses under their several headings, and also a statement of monthly balances. He shall present annually, to the Board of Direction, a balance sheet of his books as of the 31st of December, and shall furnish, from time to time, such other statements as may be required of him.

He shall conduct the correspondence of the Society and keep full records of the same.

He shall have charge of the Society's house and its contents; shall supervise the work of all employees of the Society, and see that they diligently perform their respective duties.

He shall perform all other duties which may from time to time be assigned to him by the Board of Direction.

5.—The Board of Direction may also, if they deem it necessary, appoint an Assistant Secretary, who shall aid the Secretary and be under his immediate direction in all matters. His whole time shall be given to the Society.

6.—The Secretary and Treasurer shall be paid salaries to be determined by the Board of Direction ; but such salaries shall not be reduced during the term of office, as provided in this Constitution. All other salaries shall be fixed, from time to time, by the Board of Direction.

7.—The Board of Direction shall meet within twenty days after the Annual Meeting, and shall then appoint from its members a Finance Committee of five, a Library Committee of five, and a Committee on Publications of five. At least three members of the Finance Committee, and two members of the other Committees, shall be resident within fifty miles of New York.

These Committees shall report to the Board of Direction, and perform their duties under its supervision.

8.—The Finance Committee shall have immediate supervision of the accounts and financial affairs of the Society; shall employ an expert accountant to audit the accounts monthly; shall approve all bills before payment, and shall make recommendations to the Board of Direction as to the investment of moneys, and as to other financial matters.

9.—The Library Committee shall have general supervision of the Library and the House of the Society and the property therein; shall make recommendations to the Board with reference thereto, and shall direct the expenditure for books and other articles of permanent value, of such sums as may be appropriated for these purposes.

10.—The Committee on Publications shall have general supervision of the publications of the Society, and of contracts and expenditures connected therewith.

11.—In the consideration of papers offered for presentation, those papers containing matter readily found elsewhere, those specially advocating personal interests, those carelessly prepared or controverting established facts, and those purely speculative or foreign to the purposes of the Society, shall be rejected. The Committee on Publications shall determine which papers shall be read in full, and which shall be printed after reading by title only. The Committee may return a paper to the writer for correction and emendation, and call to its aid one or more members of special experience relating to the subject treated, either to advise on the paper or to discuss it. Such papers as in the judgment of the Committee should appear in the *Transactions*,

shall promptly, upon their acceptance, be printed and distributed to members of all grades; others shall, with the consent of the authors, be suitably indexed, and filed for reference, or the Committee may provide abstracts thereof, which, when approved by the authors, may be published instead of the original papers. Advance copies of papers and discussions may be sent out to the membership before final publication.

12.—Special committees to report upon engineering subjects shall be authorized only by a majority of the votes cast by the Society, and in the following manner: A proposition to appoint such a Committee shall be presented at a regular meeting of the Society, and if sustained, on a motion to refer the same to the Board of Direction, by an affirmative vote of not less than twenty-five Corporate Members, it shall be so referred.

The Board of Direction shall then consider the same and report its recommendations to the Society at the next general business meeting, together with a statement of the arguments for and against the appointment of such Committee.

If a motion for the issue of a letter-ballot thereon receive the affirmative vote of two-thirds of the Corporate Members present, the Board of Direction shall, within thirty days thereafter, issue the letter ballot, accompanied by a statement of the arguments for and against the proposition.

A majority of a total vote of not less than one-third of the Corporate Membership of the Society shall be necessary for its adoption, whereupon the Committee so authorized shall be appointed by the Board of Direction.

MEETINGS.

Wednesday, June 1st, 1898, at 20.30 o'clock, a regular meeting will be held, at which Robert B. Stanton, M. Am. Soc. C.E., will address the Society on "The Cliff Dwellers of the Far Southwest; Their Homes, Their Agricultural and Engineering Works, and Their Military Knowledge and Art." The address will be illustrated by the stereopticon.

Wednesday, June 15th, 1898, at 20.30 o'clock, a regular meeting will be held, at which a paper by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., entitled, "Origin of the Gulf Stream and Circulation of the Waters in the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," will be presented. It was printed in the April number of *Proceedings*.

DISCUSSIONS.

Discussion on the paper by James Ritchie, M. Am. Soc. C. E., entitled, "The Construction of the Lorain Dry Dock and Shipyard of the Cleveland Ship-Building Company," which was presented at the meeting of April 20th, 1898, will be closed June 1st, 1898.

Discussion on the paper by H. N. Ogden, Jun. Am. Soc. C. E., entitled, "Flushing in Pipe Sewers," which was presented at the meeting of May 4th, 1898, will be closed June 15th, 1898.

Discussion on the paper by R. S. Buck, M. Am. Soc. C. E., entitled, "The Niagara Railway Arch," which was presented at the meeting of May 18th, 1898, will be closed July 1st, 1898.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.

			Date of Membership.
AYCRIGG, WILLIAM ANDERSON	102 Chambers St., New York City	Assoc. M. M.	May 4, 1892 May 4, 1898
BETTS, FRED KEELER	Asst. Engr., Dept. of Water Supply, New York City, Valhalla, N. Y.		May 4, 1898
DEAN, LUTHER	Taunton, Mass		May 4, 1898
EDWARDS, JAMES HARVEY	Chf. Eng. The Berlin Iron Bridge Co., East Berlin, Conn.	Jun. Assoc. M. M.	May 31, 1892 May 2, 1894 May 4, 1898

Name	Address	Grade	Date of Membership.
FOWLER, CHARLES EVAN	Chf. Eng., Youngsto'n Bridge Co., 521 Holmes St., Youngstown, O.	Jun. Assoc. M. M.	May 7, 1890 Dec. 6, 1893 May 3, 1898
HARTS, WILLIAM WRIGHT	First Lt., Corps of Engrs., U. S. A., Custom House, Frankfort, Ky.	Assoc. M. M.	Oct. 2, 1895 April 6, 1898
KUERSTEINER, EMIL EDWARD	1954 Floyd St., Louisville, Ky.		Dec. 2, 1897
LABELLE, HENRY FRANCIS	500 Bloomfield Ave., Montclair, N. J.		April 6, 1898
LEE, WELLINGTON BARNES	Chief Draftsman, Ramapo Iron Works, Hillburn, N. Y.	Assoc. M. M.	Feb. 5, 1896 May 4, 1898
McKIM, ALEXANDER RICE	Cons. Archt. Eng., 106 East 23d St., New York City.	Assoc. M. M.	April 4, 1894 May 4, 1898
MORRISON, HENRY PRENTICE	West New Brighton, N.Y.		April 6, 1898
MORSE, CHARLES ADELBERT	Res. Eng., Chicago Div., Atchison, Topeka and Santa Fé R. R., Fort Madison, Iowa		April 6, 1898
SPÖRCK, KARL	Thayer Bldg., 9th and Broadway, Kansas City, Mo.		Mar. 2, 1898

ASSOCIATE MEMBERS.

Name	Address	Grade	Date of Membership.
ALDEN, CHARLES AMES	Pennsylvania Steel Co., Steelton, Pa		May 4, 1898
ALDERMAN, CHARLES ALDO	Eau Claire, Wis.		April 6, 1898
BURNS, JUSTIN	2059 Anthony Ave., New York City		April 6, 1898
CRANE, ALBERT SEARS	47 Municipal Building, Brooklyn, N. Y.	Jun. Assoc. M.	Sept. 3, 1895 May 4, 1898

			Date of Membership.
DAVIS, CARLETON EMERSON	N. Rochester, Mass.		April 6, 1898
MOSES, JOHN CRANCH	70 Kilby St., Boston, Mass.	Jun.	July 2, 1890
		Assoc. M.	May 4, 1898
MOTT, DANIEL LIVERMORE	68 Arcade Bldg., Utica, N. Y.		April 6, 1898
PAQUETTE, CHARLES ALFRED	Eng. Maintenance of Way, Peoria and Eastern Ry., Indianapolis, Ind.		April 6, 1898
STRONG, WILLIAM EDWARD SCHENCK	Supt., Car Dept., Michigan - Peninsular Car Co., Detroit, Mich.		April 6, 1898

JUNIORS.

HARRINGTON, JOHN LYLE	Care of Cambria Iron Co., Johnstown, Pa.	Aug. 31, 1897
SWAAB, SOLOMON MARK	Bureau of Highways, City Hall, Philadelphia, Pa.	May 3, 1898

CHANGES AND CORRECTIONS.

MEMBERS.

APPLETON, THOMAS	Care of Otto Gas Engine Works, 360 Dearborn St., Chicago, Ill.
AUCHINCLOSS, WILLIAM S.	Atlantic Highlands, N. J.
BOGUE, VIRGIL GAY	66 Manhattan Life Ins. Bldg., 66 Broadway, New York City.
BURR, EDWARD	Capt., Corps of Engrs., U. S. A., 2728 Pennsylvania Ave., Washington, D. C.
CATTELL, WILLIAM ASHBURNER	Richmond Hill, Queens Co., N. Y.
CLAPP, LORENZO RUSSELL	Hempstead, Queens Co., N. Y.
CLARKE, THOMAS CURTIS	127 Duane St., Room 35, New York City.
COLEMAN, CLARENCE	U. S. Asst. Engineer, Duluth, Minn.
FOLLETT, WILLIAM W.	El Paso, Texas.
HALL, WILLIAM MCLAURINE	U.S. Engineer Office, New Bedford, Mass.
HARRISON, CHARLES LEWIS	Lansingburg, N. Y.
KEATING, EDWARD HENRY	Mgr., Toronto Ry. Co., Toronto, Ontario, Canada.
MCCULLOH, WALTER	"Arcade," Niagara Falls, N. Y.
MCCURDY, JOHN EGBERT	Apartado 26, Chihuahua, Mexico.
NEILSON, CHARLES	Rooms 24 and 25, Wyatt Bldg., Corner 14th and F Sts., N.W., Washington, D.C.
RIFFLE, ALBERT STANLEY	Care of S. F. & S. J. V. Ry., Martinez, Contra Costa Co., Cal.
SHANKS, THOMAS PEARMAN	P. O. Box 515, Louisville, Ky.
SEYMOUR, HORATIO	Marquette, Mich.
THOMAS, BENJAMIN FRANKLIN	Asst. U. S. Engineer, Mahan, Pa.

ASSOCIATE MEMBERS.

FOCHT, LOUIS ....Eng. to the State Board of Assessors, Trenton, N. J.
HARTWELL, HARRY.....................53 East 87th St., New York City.
HUTCHINSON, CARY TALCOTT...........Room 1514, 71 Broadway, New York City.
KHUEN, RICHARD, Jr..................Care of Pencoyd Iron Works, Pencoyd, Pa.
LAWLOR, THOMAS FRANCIS ....Chf. Eng. for G. F. Mellen Co., Washington Life Bldg., New York City.
LUCAS, EUGENE WILLETT VAN COURT...Care of War Department, Washington, D. C.
MCMEEKIN, CHARLES WILLIAMS ....Prin. Asst. Eng., Water Dept., Anaconda Copper Mining Co., Anaconda, Mont.
ROSENBERG, FRIEDRICH...............132 West 130th St., New York City.
SEYMOUR, HORATIO...................Marquette, Mich.
TOWSON, MORRIS SHERMAN............39 Dunham Place, Cleveland, O.

JUNIORS.

BRUSH, WILLIAM WHITLOCK...........510 Jefferson Ave., Brooklyn, N. Y.
CRAIG, WASHINGTON RIGHTER.........Asst. Supervisor, Phila. & Reading Ry., Shamokin, Pa.
DOUGLAS, FREDERICK LUKE ....St. James Bldg., 1133 Broadway, New York City.
EVANS, JOHN MAURICE................164 West 128th St., New York City.
FOLGER, EDWARD PELL.............. 80 Quincy St., Brooklyn, N. Y.
GORMLY, WALTER BRUCE..............361 West 123d St., New York City.
HAAS, EDWARD FRANCIS...............320 Sansome St., San Francisco, Cal.
HIGHLEY, LEE.......................Care of I. C. R. R. Eng. Corps, Lucy, Tenn.
LATTING, BENJAMIN FRANKLIN.........107 Sly St., Elmira, N. Y.
SELTZER, HARRY KENT................Care of Asst. Eng., P. R. R., 32d and Powelton Ave., Philadelphia, Pa.
STEVENS, PERLEY EGBERT............ 202 11th St., S. W. Washington, D. C.
SWEITZER, NELSON BOWMAN, Jr........Inspector Surveys, General Land Office, Washington, D. C.

DEATHS.

NEWHAM, CHARLES EDWARD...........Elected Member Dec. 7th, 1887; died Feb. 1st, 1898.
RICE, EDWARD CURTIS................Elected Member Apr. 7th, 1875; died Apr. 21st, 1898.
TAINTOR, WILLIAM NOYES.............Elected Junior Sept. 3d, 1895; died Apr. 8th, 1898.

ADDITIONS TO

LIBRARY AND MUSEUM.

From the American Institute of Mining Engineers:
- Apparatus for the Removal of Sand from Waste-Water of Ore-Washers.
- Mining Districts of Colombia.
- Kotchkar Gold-mines, Ural Mountains, Russia.
- Kalgoorlie, Western Australia, and Its Surroundings.
- Relation of the Strength of Wood under Compression to the Transverse Strength.
- Some Dike Features of the Gogebic Range.
- Accumulations of Amalgam on Copper Plates.
- Proceedings of the Seventy-fourth (Twenty-eighth Annual) Meeting, Atlantic City, N. J., February, 1898.

From George I. Bailey, Albany, N. Y.:
- Annual Report of the Water Commissioners of Albany, for 1897.

From Onward Bates, Chicago:
- Specifications for Steel and Iron Work, for Bridges.

From the Bignall & Keeler Manufacturing Company:
- Catalogue No. 14 of the Peerless and Duplex Pipe Threading and Cutting Machines.

From the Board of Railroad Commissioners, New York:
- Fifteenth Annual Report, for 1897, 2 Vols.

From the Civil Engineers' Club of Cleveland:
- Constitution and List of Members, March 31st, 1898.

From the Connecticut State Board of Health:
- Twentieth Annual Report, for 1897.

From Charles Corner, Austin, Texas:
- Sixth Annual Report of the Railroad Commission of the State of Texas, for 1897.

From W. Bell Dawson, Ottawa, Canada:
- Survey of Tides and Currents in Canadian Waters, Report of Progress.

From the Engineering Association of the South:
- Proceedings, Vol. IX, No. 4.

From the Engineers' Club of New York;
- Constitution, Rules, Officers and Members, 1898.

From J. B. Henderson, Brisbane, Queensland, Austria:
- Report on Water Supply, for 1897.

From Rudolph Hering, New York City:
- Indianapolis *News*, containing Report of Mr. Rudolph Hering on a Sewerage and Drainage System, June 21, 1892.
- Deterioration of Water in Reservoirs and Conduits. By Charles B. Brush.
- Sewerage and Drainage of Trenton. By Rudolph Hering.
- Relation of Ground-Water to the Health of a Community. By Geo. E. Waring, Jr.
- Confidential Report on the Head of Lake Superior as a Location for the Manufacture of Steel Rails. By W. F. Mattes, 1890.

From the Illinois Steel Company:
- An Account of the Works of the Company.

From the Institution of Civil Engineers London:
- Minutes of Proceedings, Vol. CXXXI.
- List of Members, April 1, 1898. Two copies.

From the John Crerar Library, Chicago:
- Third Annual Report, 1897.

From W. C. Kernot, Melbourne, Victoria:
- On Some Common Errors in Iron Bridge Design.

From the Königliche Technische Hochschule, Aachen, Germany:
- Feier zur Einweihung des Neubaues für Elektrotechnik und Bergbau, May 15, 1897.
- Festrede zur Vorfeier des Geburtstages des Kaisers Wilhelm II. gehalten am 26 Januar, 1898.

From Theodore A. Leisen, Engineer and Superintendent:
- Report of Board of Park Commissioners, Wilmington, Delaware, for 1897.

From the Librarian of the Engineering Society of the School of Practical Science, Toronto:
- Papers read before the Society, 1897–98.

From the Massachusetts Railroad Commissioners:
- Annual Report for 1897.

From Charles Mayne, Shanghai, China:
- Report of the Municipal Council of Shanghai for 1897 and Budget for 1898.

From Natividad Gonzalez, Mexico:
- Informe rendido a la Secretaria de Comunicaciones y Obras Publicas por el Ingeniero Edgardo K. Smoot.

From the New England Free Trade League:
- Do Protective Tariffs Give More Work to Workingmen? Charles Warren.
- Success of Free Trade. Sir Robert Griffen.

From the New York State Library:
- Annual Report for 1897.

From William Jasper Nicolls, Philadelphia:
- Coal Catechism.

From the North of England Institute of Mining and Mechanical Engineers:
- An Account of the Strata of Northumberland and Durham as proved by borings and sinkings. U–Z.

From the Pennsylvania Steel Company, Steelton, Pa.:
- General Catalogue, 1898.

From the Public Library, Cincinnati, O.:
- Annual Reports of the Librarian and Treasurer for the year ending June 30th, 1897.

From H. von Schon, Sault Ste. Marie, Mich.:
- Synopsis of Works to be Constructed at Sault Ste. Marie, Mich., by the Lake Superior Power Co. Five copies.

From the Smithsonian Institution, Washington, D. C.:
- Annual Report for the year ending June 30th, 1895.

From the Society of Engineers, London:
- Transactions for 1897.

From the Society of Naval Architects and Marine Engineers:
- Names and Addresses of the Officers and Members, January 1st, 1898.
- Transactions for 1897.

From the State Agricultural College, Fort Collins, Colo.:
- Bulletin, No. 41: Blight and the plant diseases.
- Bulletin, No. 42: Sugar beets in Colorado in 1897.

From E. H. Stone, Calcutta, India:
- Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel.

From Svenska Teknologforeningens, Stockholm:
- Förteckning. Mars. 1898.

From U. S. Chief of Engineers:
- Reservoir Sites in Wyoming and Colorado. Two copies.
- Viaduct Across Rock Creek, District of Columbia. Two copies.
- Eight Pamphlets.
- Annual Report for 1897. Vol. I, 6 parts.

From U. S. Geological Survey:
- Eighteenth Annual Report, Part V.

From the U. S. Nautical Almanac Office:
- American Ephemeris and Nautical Almanac for 1900.

From the U. S. Treasury Department, Bureau of Statistics:
- Commerce and Navigation of the United States, 1897.

From U. S. War Department:
- Report of the Chief of Ordinance for 1897. Two copies.

From the University Press, Knoxville, Tenn.:
- The University of Tennessee Record. Review of 1897.

From the West Virginia Society of Civil Engineers and Architects:
- Proceedings of the Third Annual Meeting held at Morgantown, W. Va., January 26th and 27th, 1898.

From Colonel Park Wordward, Atlanta, Ga.:
- Section of Lead Pipe showing the Effect of Electrolysis.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS:

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH.

By CHARLES D. MARX, M. Am. Soc. C. E.; CHARLES B. WING, Assoc. M. Am. Soc. C. E., and LEANDER M. HOSKINS, C. E.

TO BE PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

Through the courtesy of C. K. Bannister, M. Am. Soc. C. E., and at his suggestion, the authors were enabled to carry out, in August, 1897, a limited series of experiments on the flow of water in the recently finished conduit of the Pioneer Electric Power Company, of Ogden, Utah, of which he was at that time chief engineer. As the main dam has not been constructed, it was not possible to extend the

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

experiments to the velocities of flow corresponding to the full carrying capacity of the conduit. It is the intention to do so when opportunity offers. Believing that the results obtained so far, and a description of the method of experimentation adopted, may prove of interest to the profession, this paper is submitted.

For a description of the entire pipe line and of the construction of the pipe, reference may be made to a paper recently published.* The plan and profile of each of the portions experimented upon are given herewith, in connection with the account of the experimental results.

I.—Description of Object and Methods.

General Plan.

The main object of the experiments was to determine the relation between the rate of discharge of the pipe and the loss of head between certain definite points. To accomplish this it was necessary to measure, simultaneously, the pressure at each end of the length of pipe under experiment, and the rate of discharge.

As a secondary object, observations were taken on the loss of head in the Venturi meters, with different velocities of flow.

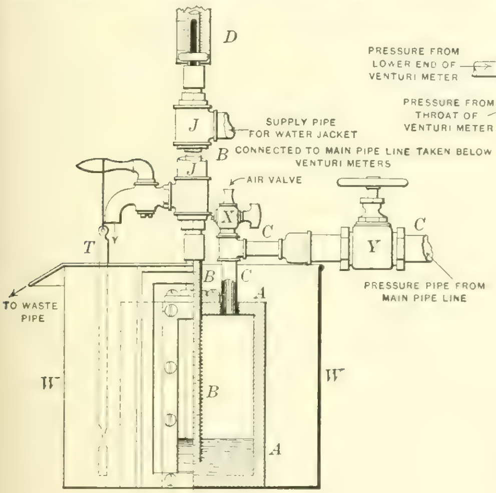
Pressure Measurements.

(1) *Method Adopted.*—A consideration of the probable range of the losses of head to be measured made it evident that a far greater degree of precision was required in the pressure measurements than could be attained with any pressure gauge of the Bourdon type. It seemed reasonably certain that the best results attainable with such a gauge would be unreliable by at least as much as 1 lb. per square inch (equivalent to a water column of 2.3 ft.); more probably the errors would be two or three times that amount, or even greater. Such a degree of uncertainty would render the results wholely valueless. It was therefore decided to use mercury gauges.†

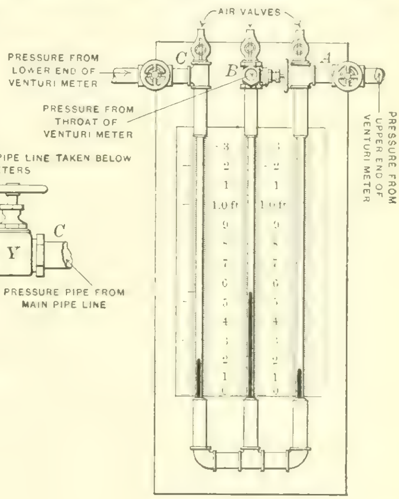
* "The Power Plant, Pipe Line and Dam of the Pioneer Electric Power Company at Ogden, Utah," by Henry Goldmark, M. Am. Soc. C. E. *Transactions*, Vol. xxxviii, p. 246.

† This discussion of Bourdon gauges has reference only to their applicability to this series of experiments. It is not meant to imply that such gauges can never give satisfactory results in experiments of this kind. For a great length of pipe the loss of head may be so great that the errors of a Bourdon gauge are relatively unimportant. In any case, however, such gauges should be repeatedly and carefully tested, during the period of experimenting, at about the pressures at which they are used. It will aid in eliminating gauge errors if the pressure is measured at various points along the profile of the pipe.

(2) *Description of Gauges.*—The form of gauge used was the ordinary open manometer. Fig. 1*a* shows the essential parts of the manometer as used at the point of greatest pressure, near the power house. Besides the manometer proper, the figure shows a water-jacket surrounding the mercury tube, and a vessel into which the water passing through the water-jacket wasted; the mercury reservoir, being placed in this vessel and being surrounded by water. The mercury reservoir *A* is of cast iron, the interior being cylindrical, 4 ins. in diameter. In one side is a window, fitted with glass, through which the position of the mercury surface may be observed. The vertical tube *B B* is of ¼-in. wrought-iron pipe, except at the upper end, where a length of



LOWER MANOMETER
Fig. 1*a*.



DIFFERENCE GAUGE
Fig. 1*b*.

glass tube *D* is attached, with a scale for reading the position of the top of the mercury column. Into the top of the reservoir *A* is fitted a pipe *C C*, leading to that cross-section of the main pipe at which the pressure is to be measured. In the pipe *C C* is a gate-valve *Y*, and at *X* is an air-cock. When the gauge is in use the pipe *C C* and the space in the reservoir above the mercury are filled with water. The air in the reservoir is removed by opening the air-cock *X* and allowing water to enter slowly through the partially open valve *Y*. The air-cock is then closed and the valve *Y* fully opened, and mercury rises in the tube *B B*. The height of the mercury column measures the water

pressure at the surface of the mercury in the reservoir; and thus, by allowing for difference of level, the pressure is known at any point of the pipe *C C* and of the cross-section of the main pipe to which this pipe is attached.

Two questions in connection with the use of the gauge call for some explanation—the effect of changes of temperature upon the mercury column, and the method of attachment of the gauge to the main pipe.

(3) *Correction for Temperature.*—In order that the observations may be comparable, the observed height of the mercury column must in every case be reduced to that of an equivalent column at some standard temperature. It was therefore necessary to know the temperature of the mercury at the time of each observation of its height. The degree of precision needed in the temperature observations may be estimated by noticing that a change of 1° Cent. affects the length of the mercury column by about $\frac{1}{5550}$ of its value. The static pressures at the four points at which measurements were made (expressed in height of water column) were about 462 ft., 153 ft., 101 ft. and 48 ft., respectively. Temperature changes of about 1.2°, 3.6°, 5.5° and 11.5° Cent. at these four points, respectively, would thus each correspond to an error of 0.1 ft. in the measured pressure-head.

At the lower end of the steel pipe, where the pressure had its greatest value, it was decided to employ a water-jacket for the purpose of keeping the mercury at a nearly constant temperature. This jacket is shown in Fig. 1*a*, already referred to. The ¼-in. mercury tube was enclosed in a larger pipe *J J*, through which a stream of water was allowed to flow continuously during the time of experimenting. The temperature of this water remained nearly constant, and was read at intervals by means of a chemical thermometer *T*, suspended in the waste-vessel *W*. This water-jacket surrounded a length of about 30 ft. of the column, the total length of which varied from 34 ft. to 32.8 ft. A length not exceeding 4 ft. was thus left unjacketed; but it is believed that no serious error was thus introduced. Thus, for a length of 4 ft. of mercury, an error of about 10° Cent. would be required to introduce an error of 0.1 ft. (water) into the pressure-head.

At the other pressure stations the heights of the mercury columns were much less, and it was thought sufficient to determine the temperature by means of a thermometer placed beside the manometer tube. During most of the observations taken at the upper end of the

steel pipe line, where the greatest height of the mercury was about 11.3 ft., the instrument was shaded during the time of experimenting. At the upper station on the wooden pipe line the manometer was fully shielded from the sun by a shed built for the protection of the pipe against rock slides, and the temperature of the air was very constant. At the lower station on the wooden pipe line the tube containing the mercury column was shielded from the sun by a 2-in. plank upon which it was mounted, and the temperature was read from a thermometer hanging beside it.

Although, in some respects, more favorable temperature conditions would have been desirable at two of the manometer stations, it is not believed that serious error was introduced into the results from this cause.

(4) *Attachment of Manometers to Main Pipe.*—The proper method of attachment of a piezometer for the purpose of measuring the pressure at any section of a pipe carrying water has been the subject of considerable discussion. The main questions raised have been: (*a*) whether a single point of attachment to the pipe is sufficient, or whether attachment should be made to a chamber communicating with the pipe at several points of the circumference; and (*b*) whether in case attachment is made only at a single point, this should be at the top or at some other point on the circumference. A careful consideration of the principles involved led the authors to the conclusion that the number of points of attachment is immaterial, and that the position of the point of attachment on the circumference is important only as affecting the liability to the collection of air in the pipe leading to the piezometer.*

In the case of the steel pipe the simplest method available was to make connection with relief valves at the top of the pipe. At the lower end the manometer was located in the power house, about 200 ft. from the point of attachment to the main pipe; the connecting pipe, $\frac{3}{4}$ in. in diameter, entering the building through a window. At the highest point of this connecting pipe was placed a valve for drawing off air. At the station near the upper end of the steel pipe line the connection with the manometer was made through a pipe about 20 ft. in length, with a valve for air at the highest point. In both cases the air valves were opened at intervals during each run, in order to remove any possible accumulation of air.

* A further discussion of these questions is given in the appendix to this paper.

In connecting the manometer to the wooden pipe, a hole was bored at right angles to the axis of the pipe. Into this hole the pipe leading to the manometer was screwed, care being taken that there should be no projection beyond the inner surface. At each of the pressure stations on the wooden pipe line the length of the connecting pipe was only a few feet.

On Plate XVII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the wooden pipe; and on Plate XVIII, Figs. 1 and 2 show the upper and lower manometers, respectively, on the steel pipe. The difference-gauges on the north and south Venturi meters, respectively, are shown by Figs. 1 and 2, Plate XIX.

Measurement of Rate of Discharge.

(*1*) *Method Adopted.*—The main pipe, 72 ins. in diameter, divides near the power house into two branches, each 54 ins. in diameter. Each of these branch pipes is furnished with a Venturi meter with automatic register. These meters were the only available means of measuring the rate of discharge with any approach to precision.

The automatic registers, however, were not adapted to the purpose in hand. The register is an integrating device, recording the total quantity discharged, but not showing the rate of discharge directly. The rate of discharge can be determined from its readings only by observing the difference between the readings at the beginning and end of a known period of time. This period must be of considerable length, since the smallest quantity directly indicated by the register is 10 000 cu. ft.; and intermediate readings cannot be estimated with precision. Moreover, the result of such a determination is not reliable unless the rate of discharge has remained uniform, since the indicating mechanism moves only once in ten minutes, registering at each movement an amount proportional to the rate of discharge at that instant. The register itself gives no indication as to what fluctuations in the rate of discharge occur during the ten-minute intervals. It was therefore decided to attach to each meter a mercury gauge, designed to show, at every instant, the difference between the pressures existing at the throat and the up-stream sections of the Venturi. This pressure difference indicates the rate of discharge instantaneously; no time interval need be measured.

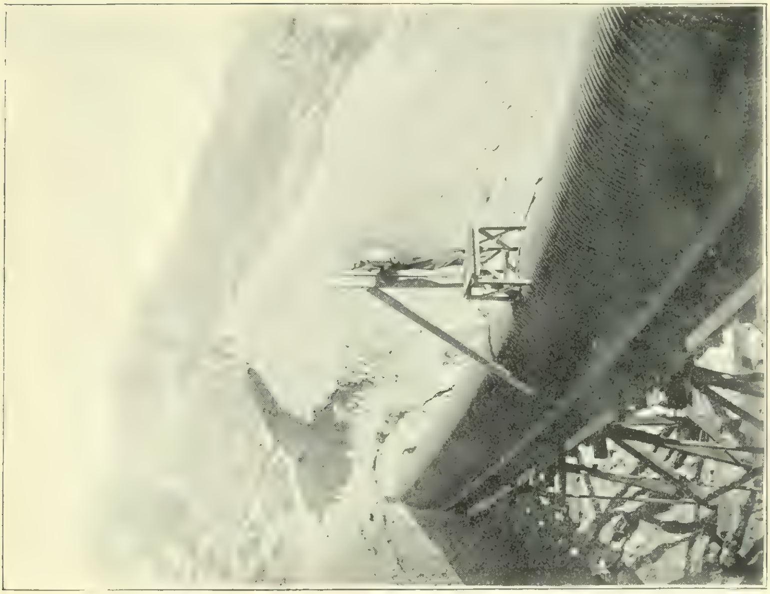


Fig. 2.

Fig. 1.

(2) *Difference-Gauges.*—For accomplishing the object stated, the essential parts of the difference-gauge are two vertical glass tubes connected at the bottom, and at the top communicating respectively with the two sections whose pressure difference is to be determined. The tubes are filled with mercury to a convenient height, the space above the mercury and the connecting pipes being filled with water. By means of a fixed vertical scale, the difference in level between the tops of the two mercury columns may be read at any desired instant.

The gauges, as actually constructed, were designed to accomplish an additional object, namely, to indicate the total loss of head in the meters. For this purpose a third tube was added, connected with the other two at the lower end, and at the top communicating with the 54-in. pipe immediately below the meter. Simultaneous readings of the three mercury columns thus served to determine both (*a*) the instantaneous rate of discharge, and (*b*) the corresponding loss of head in the meter.

In Fig. 1*b* are shown the essential features of these difference-gauges as actually constructed and used. The three glass tubes communicate with each other freely at the bottom through suitable connections of wrought-iron pipe. At the top they communicate with separate pipes *A*, *B*, *C*, leading respectively to the three sections whose pressures are to be compared. Each is furnished at the highest point with an escape cock for drawing off air, and in each pipe is placed a gate-valve.

In attaching the gauge for use, the air-valves *V* were at first left open, and the space in the tubes above the mercury completely filled with water, the gate-valves in the connecting pipes being, for this purpose, only slightly opened. These latter valves were tightly closed as soon as all the air had escaped from the tubes and connecting pipes, and very gradually opened again after the air-valves had been closed. No difficulty was experienced from air in the pipes *A*, *B*, *C*. In two of these pipes (*A* and *B* in each gauge) the air-valve was at the highest point; in the third (that running to the section below the meter) this was not the case. Whether air was present in any pipe could be tested at times of no discharge, since under static conditions the three mercury columns must stand at the same level if the connecting pipes were completely filled with water. During the series of experiments there were frequent opportunities of applying this test, and air was never found to be present, even in the pipe with the upward bend.

except in two or three instances. These instances occurred after the adjacent receiving chamber had for some reason been emptied of water; and it was always found easy to blow off the air through the air-valve *V*.

The scales for reading the heights of the mercury in the tubes were graduated to hundredths of a foot, and the third decimal place was estimated in taking readings. In setting up the instrument, the adjustment was determined by the equality of the readings of the three scales under static conditions.

Programme of Tests.

In carrying out the experiments, an observer stationed at each of the gauges in use took readings at short intervals during a certain time previously agreed on. The interval between readings was originally intended to be one minute, but it was found that under steady conditions it was not always necessary to read so frequently.

The rate of discharge was controlled by the superintendent in charge of the power house. The aim was to maintain a uniform flow for a certain period; then to change the rate of discharge and maintain a uniform flow for another period; then change again; and so on as long as desired. In this way was obtained a series of simultaneous sets of values of the three quantities sought, the rate of discharge and the pressures at two sections of the main pipe.

It was not necessary to note the time of reading with great precision, so long as nearly uniform conditions of discharge and pressure could be maintained. A sufficient degree of uniformity was secured without difficulty, except in two or three instances to be mentioned later.

After a change in the rate of discharge, some time always elapsed before a steady condition was re-established. It was found that about fifteen minutes must be allowed for each change. The first day's results were unsatisfactory because sufficient time was not allowed for steady conditions to be re-established after changes in the rate of discharge. During the period of adjustment from one steady condition to another, the pressure gauges showed marked fluctuations, the range being at first considerable,* but decreasing and nearly disappearing

*Although no accurate determination was made of the increase of pressure caused by sudden stoppage or change of the rate of discharge, it is certain that, even at the lower end of the line, the mercury column in no case rose more than 3 ft. above the static height. In one case a very sudden stoppage occurred, owing to the blowing out of a fuse, but the above statement holds even for this extreme case.

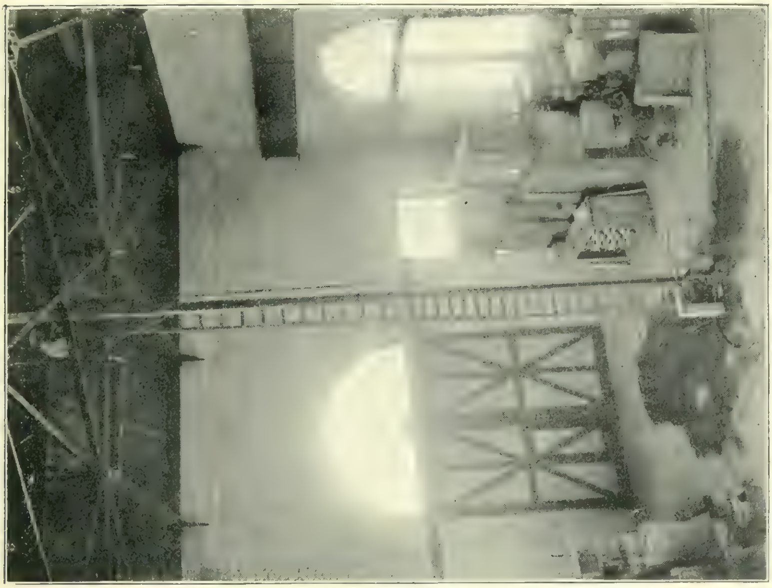


FIG. 2.

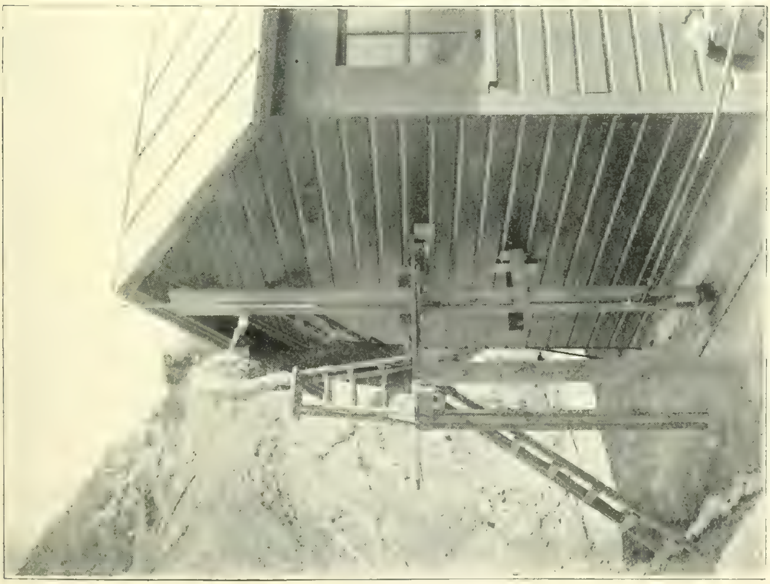


FIG. 1.

after ten or fifteen minutes. The difference-gauges attached to the Venturi meters, on the other hand, adjusted themselves to changed conditions much more quickly, their oscillations at times of change continuing not more than two or three minutes. This showed that the rate of discharge became steady very quickly, while the pressure continued to oscillate for some time. It was at first suggested that the observed oscillations of the manometer columns might be due to the inertia of the mercury. It was found, however, by plotting curves showing the simultaneous readings of the two gauges in use at the same time, that there was always a close agreement between the periods of oscillation of the two columns. This is shown in Fig. 5. It would therefore appear that the observed oscillations correspond to true oscillations of pressure in the pipe. Doubtless these variations of pressure are transmitted in waves throughout the length of the pipe. No difference of phase between the oscillations of pressure at the two stations could be detected.*

Because of conditions existing at the inlet, the highest rates of discharge observed were accompanied by a decrease of pressure throughout the pipe. Under such circumstances it was of more importance to secure exact coincidence in the times of observing the two pressures, since any uncertainty as to what observations at the two stations are simultaneous introduces an uncertainty into the value of the loss of head between the stations. It was found that the gauges at the two stations always gave practically identical determinations of the rate of decrease of the pressure. This is illustrated by the plotted results shown in Fig. 4.

It will be noticed that the oscillations of pressure at the beginning of the period represented by the curves in Fig. 5 show a period between successive maxima or minima of about six minutes. The records for the entire series of experiments show a period lying between five and six minutes. It should be said that there were also minor oscillations of a period so short that no attempt was made to record them. The period of these minor oscillations was not determined, but it was only a few seconds.

In cases of falling pressure the oscillations recorded at the two stations serve as a check upon the coincidence of the instants of observation.

* The velocity of propagation of a pressure wave should, in fact, be practically equal to the velocity of sound in water—about 4 700 ft. per second. The interval occupied in traversing the distance between the two manometers would therefore be less than one second, since the greatest length of pipe experimented upon was 4 427 ft.

Reduction of Observations.

(1) *Reduced Height of Manometer Column.*—The quantity directly given by a manometer reading was the height of the top of the mercury column above the zero of the scale. To this must be added the height of the zero of the scale above the surface of the mercury in the reservoir. This latter quantity need not be determined with great accuracy, since by the method adopted for computing loss of head (to be explained presently) a constant added to all the observed readings does not affect the final result. Except for the necessity of applying the temperature correction, the direct readings of the scale could be used instead of the total heights of the mercury column. For the purpose of applying the corrections for temperature, the height of the zero of the scale above the surface of the mercury in the reservoir was taken as a constant, its value being measured under conditions of static pressure. The reduction to an equivalent column at 0° Cent. is made by multiplying the observed height of the column by the factor 1—0.000 18 T; T being the temperature of the mercury at the time of observation, expressed in degrees Centigrade.

Strictly speaking a correction must also be made for the rise of the surface of the mercury in the reservoir. The interior diameter of the reservoir being about sixteen times that of the glass tube, a fall of the mercury in the latter must be accompanied by a rise about $\frac{1}{256}$ as great in the former. The true shortening of the column is therefore equal to the drop of the column multiplied by $\frac{257}{256}$, or about 1.004. For the greatest loss of head measured in the entire series of experiments (about 3.4 ft.), this correction would amount to only 0.014 ft. (water).

(2) *Plotting of Reduced Readings.*—After each day's run, the reduced heights of the manometer columns and the readings of the Venturi difference-gauges were plotted as ordinates with time as abscissas. By inspection of the resulting curves it was possible to select periods during which the conditions of pressure and discharge remained steady. The data corresponding to each period were averaged, and the resulting average values of the manometer heights and difference-gauge readings were regarded as constituting an "observation." Samples of the plotted results are shown in Figs. 4 and 5 which will be referred to again.

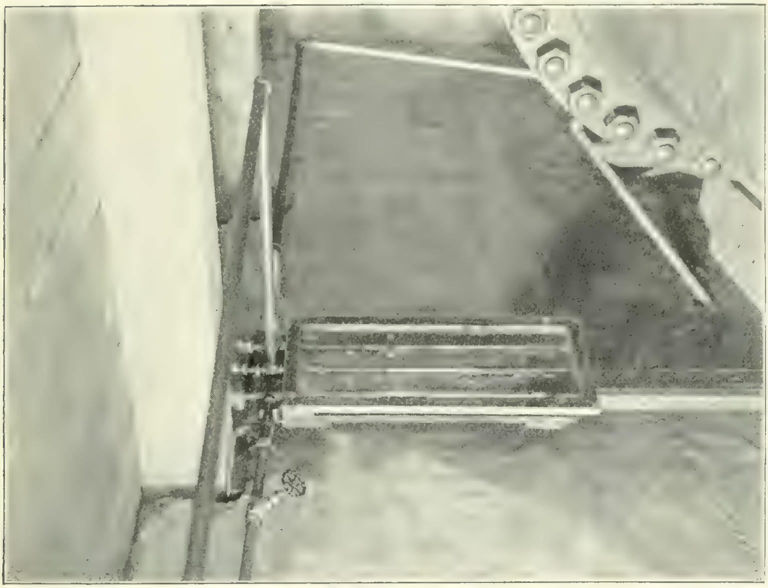


Fig. 2.

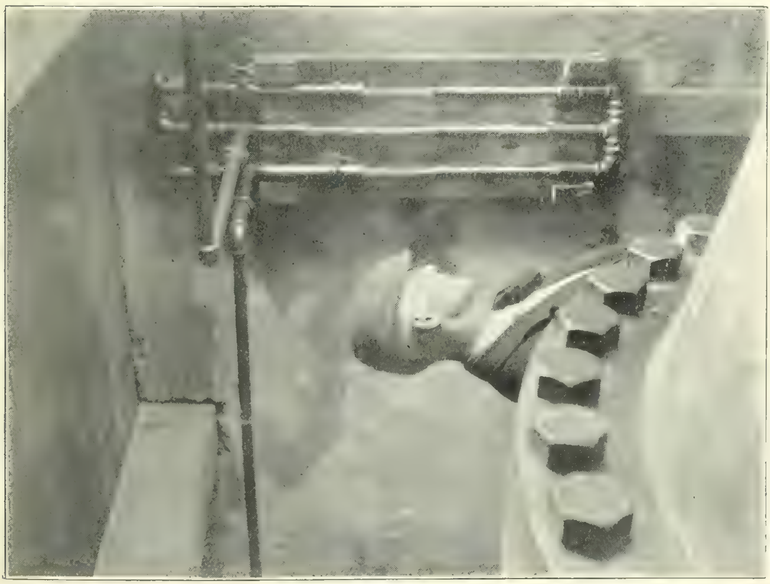


Fig. 1.

(*3*) *Computation of Loss of Head.*—Since the pipe had the same diameter at the two pressure stations, the loss of head between these stations is found by a comparison of the pressures.

Let suffixes ($_1$) and ($_2$) refer to up-stream and down-stream stations respectively, and let

m_1 and m_2 denote any simultaneous heights of the mercury columns ;

M_1 and M_2 values of m_1 and m_2 under static pressure ;

$y = m_1 - m_2$;

$Y = M_1 - M_2$;

e = specific gravity of mercury = 13.6 ;

H' = loss of head between the two stations (expressed in height of water column).

Then $e\,(M_1 - m_1)$ = loss of head in pipe from inlet to upper station ;

$e\,(M_2 - m_2)$ = " " " " lower "

$e\,[\,(M_2 - m_2) - (M_1 - m_1)\,] = e\,(y - Y)$ = loss of head between the two stations .... (1)

That is, $H' = e\,(y - Y)$ .... (2)

It is evident that the value of H' is not changed by any constant error affecting all values of m_1 (including the static value), nor by a constant error affecting all values of m_2. This justifies the statement above made that great accuracy is not required in the determination of the height of the zero of the manometer scale above the surface of the mercury in the reservoir. Any constant length of the mercury column could be disregarded, except for the necessity of reducing for temperature.

(*4*) *Computation of Rate of Discharge.*—The theory of the Venturi meter is based upon the assumption that the velocity of flow is uniform throughout each of the cross-sections whose pressures are compared. On this assumption, and neglecting loss of head between the two sections, the rate of discharge is given by the following formula :

Let Q denote the rate of discharge in cubic feet per second, computed on the above assumption, and let the difference between the pressure-heads at points on the same level in the two cross-sections (called by Herschel "head on Venturi") be represented by H (expressed in feet of water column). Then, if a_1 and a_2 are the areas of the two cross-sections,

$$Q = \frac{a_1\,a_2}{\sqrt{a_1^2 - a_2^2}}\sqrt{2\,g\,h} \qquad\qquad (3)$$

If q denotes the true value of the rate of discharge,

$$q = kQ \quad \text{.............................(4)}$$

k being a coefficient whose value is known only by experiment.

In the Ogden meters the diameters of the two cross-sections are 54 ins. and 25.5 ins., respectively. The formula therefore becomes

$$q = 3.639\, k\sqrt{2\, g\, H}; \quad \text{......................(5)}$$

q being in cubic feet per second, H in feet, g in foot-second units, and k being an abstract number.

In order to save labor in the application of the formula it is convenient to replace H by the difference in height of the two mercury columns of the difference-gauge. If h denotes this difference and e the specific gravity of mercury,

$$H = (e - 1)h; \quad \text{.........................(6)}$$

since the difference between the mercury heights is in part balanced by an equal and opposite difference between the heights of the water columns above the mercury. Taking 13.6 as the value of e, equation (5) may be written

$$q = 103.58\, k\sqrt{h} \quad \text{.......................(7)}$$

The values of k to be used with the Ogden meters were furnished to the writers, at the request of Clemens Herschel, M. Am. Soc. C. E., by the manufacturers, the Builders' Iron Foundry, Providence, R. I.

If

$$k = \frac{a_1}{\sqrt{a_1^2 - a_2^2}}\, k^1 \quad \text{.....................(8)}$$

the values of k^1 are given in the following table:

Head on Venturi. H (feet).	Coefficient. k^1.	Head on Venturi. H (feet).	Coefficient. k^1.
1	.995	13	.977
2	.992	14	.9765
3	.9895	15	.9755
4	.9875	16	.975
5	.986	17	.974
6	.985	18	.973
7	.9835	19	.9725
8	.9825	20	.9715
9	.9815	21	.971
10	.9805	22	.9705
11	.9795	23	.970
12	.9785		

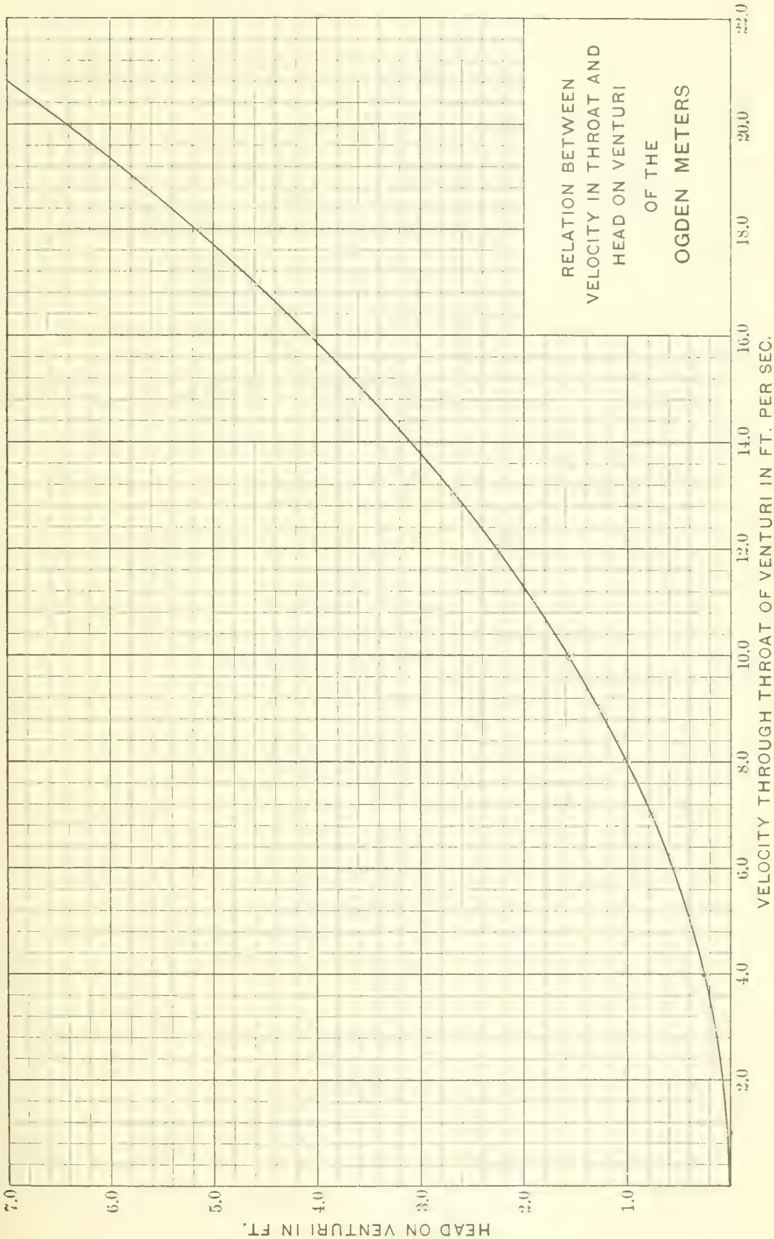

RELATION BETWEEN
VELOCITY IN THROAT AND
HEAD ON VENTURI
OF THE
OGDEN METERS
VELOCITY THROUGH THROAT OF VENTURI IN FT. PER SEC.
HEAD ON VENTURI IN FT.
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0

Fig. 2.

These values are based upon experiments made by Herschel in 1891 with the 48-in. meter of the East Jersey Water Company; the ratio of throat diameter to main diameter being the same for this meter as for the Ogden meters.

The values of the throat velocity for different values of "head on Venturi," corresponding to the above values of the coefficient k^1, are shown in Fig. 2. This diagram is copied from one furnished by the Builders' Iron Foundry, which was used in making the actual computations.

The heights of the mercury columns of the difference-gauges were read to 0.001 ft., and it is believed that the probable error in a value of h does not exceed 0.002 ft. The corresponding error in the value of q varies with q, being less as q is greater. From the relation between q and h above given, taking $k = 1$ (which is sufficiently correct for the present purpose),

$$d\,q = \frac{5360}{q}\,d\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

An error $\delta\,h = 0.002$ causes an error $\delta\,q = \frac{10.7}{q}$, which varies from 1.1 to 0.11 as q varies from 10 cu. ft. per second to 100 cu. ft. per second. The smallest values of q in the experiments under consideration were about 14 cu. ft. per second; the uncertainty of these values, so far as due to errors in the observed values of h, is probably about 5 per cent. The observational error in values of q greater than 35 cu. ft. per second is probably less than 1 per cent.

General Results.

In order to facilitate comparison with the results of other experiments, the results are represented in three ways:

(1) The loss of head per 1 000 ft. has been computed from each observation, and the relation between this loss and the velocity of flow represented graphically.

(2) The values of c in the Chezy formula,

$$v = c\sqrt{r\,s} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

has been computed for each observation, and the relation between r and c represented graphically.

(3) The value of f in the formula for loss of head in a pipe of diameter d and length l,

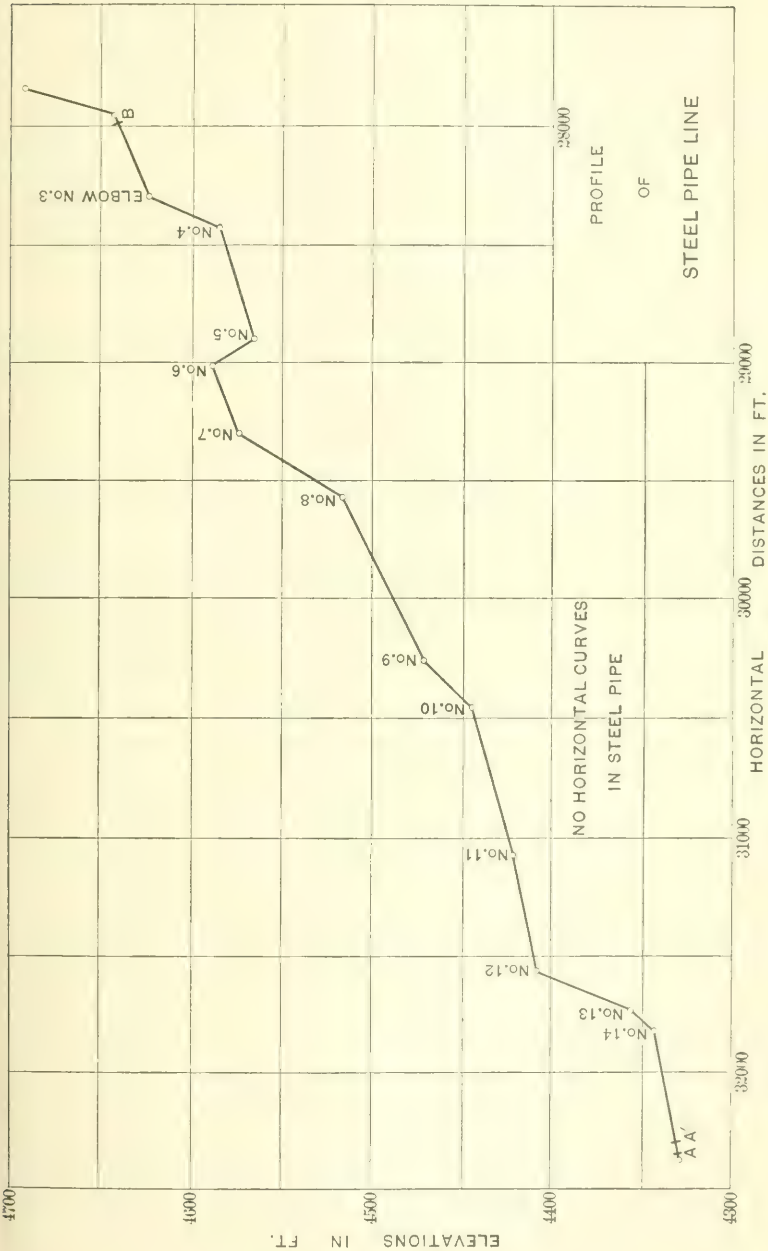

PROFILE
OF
STEEL PIPE LINE
NO HORIZONTAL CURVES
IN STEEL PIPE
HORIZONTAL DISTANCES IN FT.
ELEVATIONS IN FT.
28000
29000
30000
31000
32000
4700
4600
4500
4400
4300
B
ELBOW No.3
No.4
No.5
No.6
No.7
No.8
No.9
No.10
No.11
No.12
No.13
No.14
A A′

Fig. 3.

$$H' = f \frac{l}{d} \frac{v^2}{2g} \qquad (11)$$

has been computed for each observation, and the relation between f and v represented by a diagram.

There is a definite relation between c and f. In the Chezy formula the value of s, the hydraulic slope, is $\frac{H'}{l}$ and the value of the hydraulic radius r is $\frac{d}{4}$. The above two formulas are therefore identical if

$$f = \frac{8g}{c^2} \qquad (12)$$

Loss of Head in Venturi Meter.

As already stated, the difference-gauges attached to the Venturi meters were designed to show the differences between the pressures at three sections: (1) just before the contraction of the stream, the diameter being 54 ins.; (2) at the throat, the diameter being 25.5 ins., and (3) just below the expanding stream, the diameter being 54 ins. The difference between the pressures at the first and third sections shows the loss of head caused by friction, and by the contraction and expansion of the stream in passing through the meter. The relation between this loss and the rate of flow through the meter is represented both graphically and in tabular form.

Some trouble was experienced from the presence of dirt at the top of the mercury columns of the difference-gauges. It fortunately happened that this difficulty was confined, in both gauges, to the tube communicating with the section below the meter (the tube marked C in Fig. 1b), so that the estimate of rate of discharge was not affected. The observations for loss of head in the meter were, however, rendered less accurate than they would otherwise have been. The uncertainty in the reading of the mercury column probably in no case exceeded 0.004 ft., but this was sufficient to render valueless certain of the observations taken with low velocities of flow. In spite of this, the entire series of observations serves to show pretty satisfactorily the relation between loss of head in meter and velocity of flow, within the range of the experiments. The discussion of results follows.

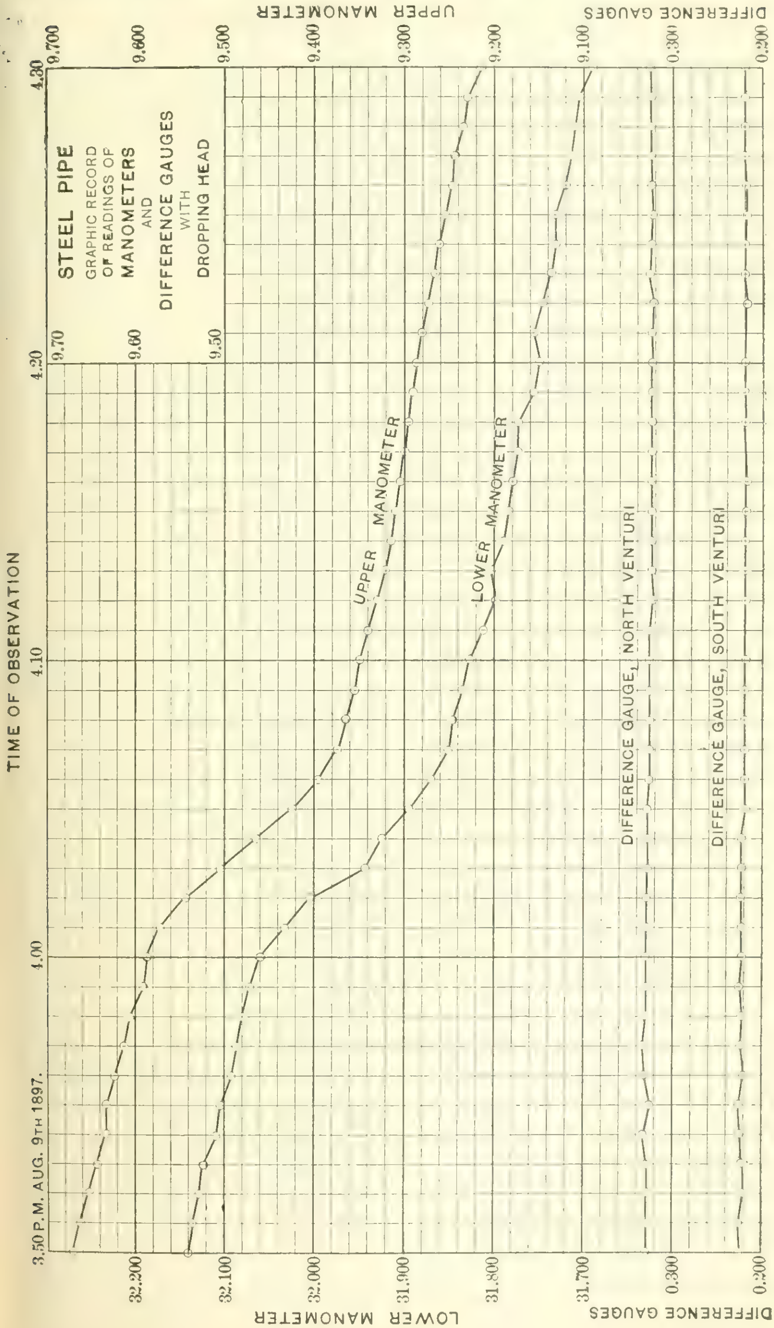

STEEL PIPE
GRAPHIC RECORD OF READINGS OF MANOMETERS AND DIFFERENCE GAUGES WITH DROPPING HEAD
TIME OF OBSERVATION
3.50 P.M. AUG. 9TH 1897.
4.00
4.10
4.20
4.30
UPPER MANOMETER
LOWER MANOMETER
DIFFERENCE GAUGES
DIFFERENCE GAUGE, NORTH VENTURI
DIFFERENCE GAUGE, SOUTH VENTURI
9.700
9.600
9.500
9.400
9.300
9.200
9.100
9.70
9.60
9.50
32.200
32.100
32.000
31.900
31.800
31.700
0.300
0.200

FIG. 4.

II.—Experiments on Steel Pipe.

Description of Pipe Line.

For a full description of the dimensions and construction of the riveted steel pipe, reference may be made to the paper previously cited.* The portion experimented upon included nearly the entire length of the steel pipe. In plan this pipe is straight; the profile is shown in Fig. 3. The positions of vertical curves or elbows are also shown in Fig. 3, the radius of curvature and total angle in each case being given in the following table:

Elbow No.	Station.	Angle.	Radius in feet.
3	283	13° 30′	30
4	284 + 80	14° 23′	30
5	289	14° 8′	30
6	290 + 10	14° 46′	30
7	293	8° 58′	30
8	295 + 70	8° 8′	30
9	302 + 50	3° 20′	30
10	304 + 50	4° 49′	30
11	310 + 70	1° 0′	30
12	315 + 81.6	17° 15′	40
13	316 + 71.6	13° 39′	30
14	318 + 31.6	7° 25′	30

"The change of direction between successive sections of an elbow was limited to 4° or 5°, so that each section is about 2½ or 3 ft. long. * * * The butt straps, sizes of rivets and rivet spacing are the same as for straight pipe."†

Throughout the series of observations on the steel pipe the upper pressure station was at the point marked B on the profile. The lower station, near the power house, was at A during observations 1–12, after which it was at A'. The length of pipe from A to B is 4 427 ft.; from A' to B, 4 367 ft. The diameter of the pipe varies slightly in different portions, the mean value being 72.22 ins.‡

Record of Observations.

(1) *Examples of Full Data Upon Which Tabulated Results Are Based.*—As already explained, the quantities recorded as constituting a single "observation" are in every case based upon readings taken at short intervals during a period of some length. These periods were selected after plotting, to a time base, the values of the four quantities ob-

* *Transactions*, Vol. xxxviii, p. 246.

† *Transactions*, Vol. xxxviii, p. 262.

‡ *Transactions*, Vol. xxxviii, p. 259.

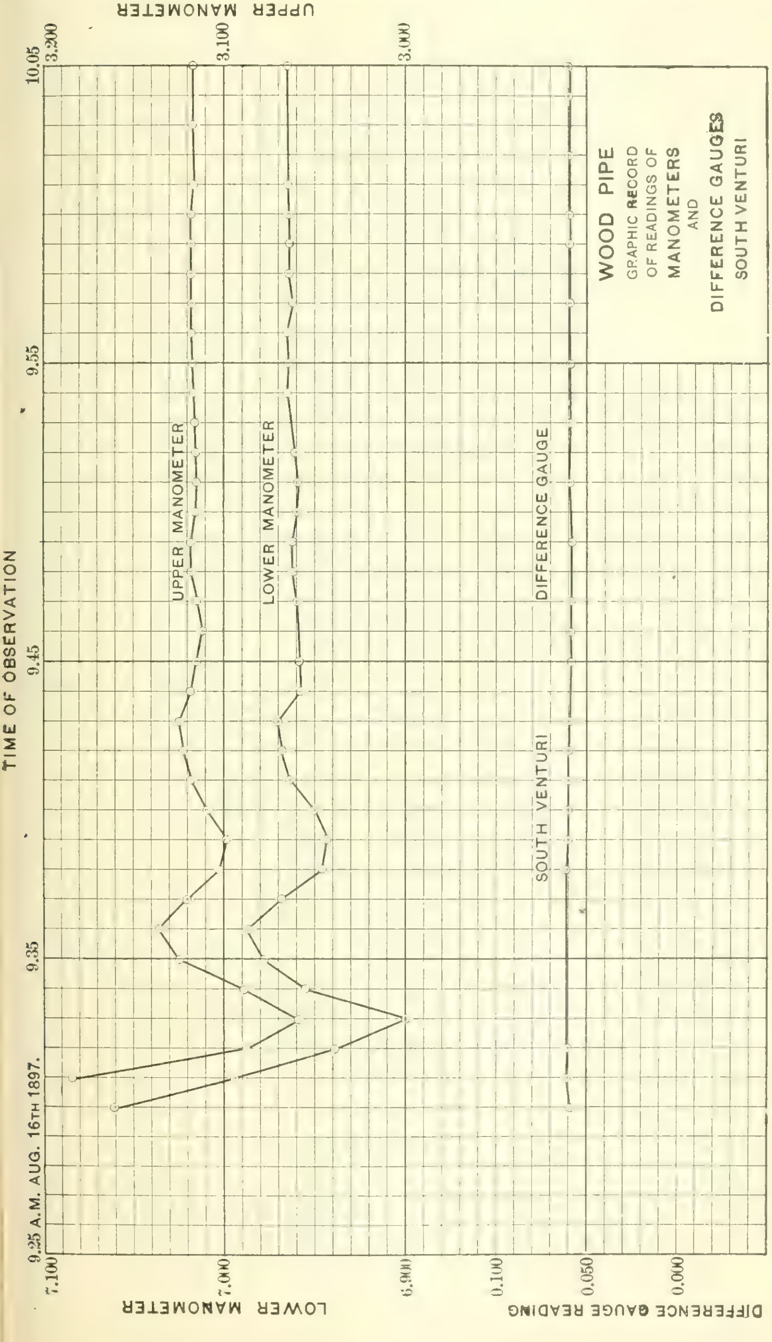

WOOD PIPE
GRAPHIC RECORD
OF READINGS OF
MANOMETERS
AND
DIFFERENCE GAUGES
SOUTH VENTURI
TIME OF OBSERVATION
9.25 A.M. AUG. 16TH 1897.
9.35
9.45
9.55
10.05
UPPER MANOMETER
LOWER MANOMETER
DIFFERENCE GAUGE
SOUTH VENTURI
DIFFERENCE GAUGE READING
3.200
3.100
3.000
7.100
7.000
6.900
0.100
0.050
0.000

Fig. 5.

served, namely, the heights of the two manometer columns (reduced for temperature), and the readings of the two difference-gauges attached to the Venturi meters (at times, however, only one meter was in use). The nature of the actual records obtained will be sufficiently indicated by showing samples of these plotted results. These are given in Figs. 4 and 5. Although the record given in Fig. 5 was taken during an experiment on the wooden pipe, it is representative of the character of the records obtained in both sets of observations, in all cases except the nine observations taken under conditions of falling pressure; while the record shown in Fig. 4 is representative of these nine observations. In these cases of falling pressure, periods were selected during which the difference-gauges showed the rate of discharge to be nearly uniform, while the manometer readings showed a nearly constant rate of decrease of the pressure at each pressure station. This rate of decrease at any given instant had practically the same value at the two stations, and the difference between the heights of the manometer columns thus remained constant during the time of an observation. In case of observations made under conditions of decreasing pressure, any discrepancy between the times of taking manometer readings at the two stations introduces an error into the observed value of the difference between heights of manometer columns. The instants of reading were as nearly coincident as they could be made by comparison of watches, and it is believed that the results are not affected by serious error. Assuming, as above, that the time of transmission of a pressure-wave between the two stations is less than one second, the oscillations of pressure furnish a check on the coincidence of the times of observation.

(2) *Tabulation of Reduced Observations.*—The entire series of experiments made upon the steel pipe resulted in twenty-nine "observations" whose periods were selected in the manner above described. The dates of these observations are shown in Tables Nos. 1 and 2.

The date and period of each observation are shown in columns 2 and 3 of Table No. 1. Columns 4 to 9 of the same table give the results of the observations on the Venturi meters. The "reading of difference-gauge," given in columns 4 and 6 for the two meters, is the difference between the observed heights of the two mercury columns connected respectively with the throat and the up-stream section of the Venturi. The equivalent water column ("head on Venturi") is

TABLE No. 1.—STEEL PIPE DISCHARGE OBSERVATIONS.

1	2	3	4	5	6	7	8	9
			North Venturi Meter.		South Venturi Meter.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Reading of difference-gauge. (Feet.)	Rate of discharge (cubic feet per second).	Rate of discharge of main pipe (cubic feet per second).	Mean velocity of flow in main pipe (feet per second).
	1897.							
1....	Aug. 4	2.35— 2.45	....	....	0.022	14.5	14.5	0.510
2....	" 4	3.23— 3.31	....	....	.199	44.8	44.8	1.575
3....	" 6	2.10— 3.00	0.022	14.5	....	....	14.5	0.510
4....	" 6	3.15— 3.40	.093	30.9	....	....	30.9	1.086
5....	" 6	3.50— 4.15	.090	30.3	....	....	30.3	1.065
6....	" 7	1.33— 1.56	....	....	.225	47.5	47.5	1.670
7....	" 7	1.57— 2.15	....	....	.225	47.5	47.5	1.670
8....	" 7	2.45— 2.53	....	....	.512	71.2	71.2	2.503
9....	" 7	3.15— 3.29	.002	4.6*	.481	69.2	73.8*	2.594*
10....	" 7	4.11— 4.23	.181	42.6	.391	62.5	105.3	3.701
11....	" 7	5.00— 5.13	.078	28.2	.141	37.6	65.8	2.313
12....	" 7	5.15— 5.50	.077	28.1	.140	37.5	65.6	2.306
13....	" 9	1.42— 2.15	.127	35.8	.296	54.5	90.3	3.174
14....	" 9	2.36— 3.01	.090	30.3	.226	47.5	77.8	2.735
15....	" 9	3.26— 4.00	.333	57.8	.221	47.1	104.9	3.687
16....	" 9	4.12— 4.30	.323	57.0	.219	46.9	103.9	3.652
17....	" 11	8.40— 8.51	.021	14.1	....	....	14.1	0.496
18....	" 11	8.55— 9.15	.021	14.1	....	....	14.1	0.496
19....	" 11	9.30—10.00	.099	31.7	....	....	31.7	1.114
20....	" 11	10.39—10.43	.095	31.2	.043	21.0	52.2	1.835
21....	" 11	11.10—11.31	.086	29.6	.159	40.0	69.6	2.447
22....	" 11	11.35—12.00	.087	29.8	.160	40.1	69.9	2.457
23....	" 12	9.15— 9.28	.204	45.5	....	....	45.5	1.599
24....	" 12	9.29— 9.59	.205	45.7	....	....	45.7	1.606
25....	" 13	8.30— 8.50	....	....	.026	16.0	16.0	0.562
26....	" 13	9.13— 9.40	.111	33.7	.072	27.2	60.9	2.141
27....	" 13	10.07—10.40	.270	51.8	.123	35.3	87.1	3.062
28....	" 13	11.36—11.46	.401	63.2	.211	46.2	109.4	3.846
29....	" 13	11.48—12.03	.400	63.1	.210	46.1	109.2	3.839

* The rate of the discharge of the N. meter was so small during this observation that it could not be accurately measured.

found by multiplying the tabulated number by 12.6 or $e - 1$, e being the specific gravity of mercury. The values of the head on Venturi are not given in Table No. 1, but will be found in Tables Nos. 8 and 9. Columns 5, 7, 8 and 9 need no explanation. In computing values of the mean velocity in the main pipe, the diameter has been taken as 72.22 ins.

The manometer observations, with the values of the loss of head deduced from them, are recorded in Table No. 2. The headings of the columns explain sufficiently the meanings of the quantities tabulated. Column 2 gives the length of the main pipe between the points of at-

TABLE No. 2.—STEEL PIPE.—MANOMETER OBSERVATIONS.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	HEIGHTS OF MANOMETER COLUMNS REDUCED TO 0° CENT. (FEET.)			Static difference of mercury columns reduced to 0° cent. (Feet.)	LOSS OF HEAD BETWEEN MANOMETERS.		Loss of head per 1 000 ft. (in feet of water).
		Upper.	Lower.	Difference.		Mercury (Feet.)	Water. (Feet.)	
1..	4 427	11.251	33.916	22.665	22.668	0.003	0.041	0.0092
2..	4 427	11.059	33.686	22.627	22.668	.041	.558	.126
3..	4 427	11.258	33.914	22.656	22.668	.012	.163	.0369
4..	4 427	11.201	33.847	22.646	22.668	.022	.299	.0676
5..	4 427	11.204	33.847	22.643	22.668	.025	.340	.0768
6..	4 427	11.018	33.661	22.643	22.698	.055	.748	.169
7..	4 427	11.013	33.659	22.646	22.698	.052	.707	.160
8..	4 427	10.590	33.171	22.581	22.698	.117	1.591	.359
9..	4 427	10.559	33.136	22.577	22.698	.121	1.646	.372
10..	4 427	Dropping	Dropping	22.453	22.698	.245	3.332	.753
11..	4 427	10.720	33.300	22.580	22.698	.118	1.605	.363
12..	4 427	10.733	33.316	22.583	22.698	.115	1.564	.353
13..	4 367	10.296	32.816	22.520	22.707	.187	2.543	.582
14..	4 367	10.559	33.113	22.554	22.707	.153	2.081	.477
15..	4 367	Dropping	Dropping	22.472	22.707	.235	3.196	.732
16..	4 367	"	"	22.472	22.707	.235	3.196	.732
17..	4 367	11.260	33.960	22.700	22.707	.007	.095	.0218
18..	4 367	11.262	33.966	22.704	22.707	.003	.041	.0093
19..	4 367	11.207	33.896	22.689	22.707	.018	.245	.0560
20..	4 367	11.022	33.669	22.647	22.707	.060	.816	.187
21..	4 367	10.754	33.352	22.598	22.707	.109	1.482	.340
22..	4 367	10.748	33.342	22.594	22.707	.113	1.537	.352
23..	4 367	11.080	33.752	22.672	22.707	.035	.476	.109
24..	4 367	11.086	33.750	22.664	22.707	.043	.585	.134
25..	4 367	11.261	33.963	22.702	22.707	.005	.068	.0156
26..	4 367	10.890	33.520	22.630	22.707	.077	1.047	.240
27..	4 367	10.375	32.923	22.548	22.707	.159	2.162	.495
28..	4 367	Dropping	Dropping	22.458	22.707	.249	3.386	.776
29..	4 367	"	"	22.460	22.707	.247	3.359	.769

tachment of the two manometers. The point of attachment of the upper manometer remained unchanged during the experiments ; that of the lower was changed after observation No. 12.* Column 6 gives the difference between the heights of the two manometer columns under static conditions. The observed value of this difference changed because the quantity of mercury in the lower manometer changed, for reasons to be considered presently. The numbers in column 7 are obtained from those in columns 5 and 6 by subtraction. The values in column 7 are reduced to equivalent water heights (column 8) by multiplying by 13.6 ; and the loss of head per thousand feet (column 9) is found from total loss of head (column 8) and length of pipe (column 2).

* The reason for this change was a suggestion that the point of attachment was at first too near the fork in the main pipe. The results, however, show no evidence of vitiation from this cause.

TABLE No. 3.—Steel Pipe. General Results.

1	2	3	4	5
No.	Mean velocity in main pipe. Feet per second.	Loss of head per 1 000 feet. (Feet.)	Value of c in formula $v = c\sqrt{rs}$.	Value of f in formula $H^1 = f \cdot \frac{l}{d} \cdot \frac{v^2}{2g}$.
17	.496	0.0218	87	0.0343
18	.496	.0093	132	.0147
1	.510	.0092	137	.0137
3	.510	.0369	68	.0549
25	.562	.0156	116	.0191
5	1.065	.0768	99	.0262
4	1.086	.0676	108	.0222
19	1.114	.0560	121	.0174
2	1.575	.126	114	.0197
23	1.599	.109	125	.0165
24	1.606	.134	113	.0201
6	1.670	.169	105	.0235
7	1.670	.160	108	.0222
20	1.835	.187	109	.0215
26	2.141	.240	113	.0202
12	2.306	.353	100	.0257
11	2.313	.363	99	.0262
21	2.447	.340	108	.0219
22	2.457	.352	107	.0226
8	2.503	.359	108	.0222
9	2.594	.372	110	.0214
14	2.735	.477	102	.0246
27	3.062	.495	112	.0204
13	3.174	.582	107	.0224
16	3.652	.732	110	.0212
15	3.687	.732	111	.0208
10	3.701	.753	110	.0213
29	3.839	.769	113	.0202
28	3.846	.776	113	.0203

(3) *General Results.*—In Table No. 3 are shown three sets of results, arranged for convenient comparison with other recorded experiments and accepted results. The observations are arranged according to values of v, the mean velocity of flow (column 2), and in columns 3, 4 and 5 are given the values of the three quantities most commonly employed in estimating the carrying capacity of pipes. The quantities given in this table are also represented graphically in Figs. 6, 7 and 8.

Fig. 6 shows the loss of head per thousand feet as a function of the mean velocity of flow. The curve expressing this relation would be a parabola with vertex at the origin, and principal axis parallel to the ordinates representing loss of head, if c and f were constants.

In Fig. 7 each observation is represented by a point whose abscissa is v, and whose ordinate is the value of c in the Chezy formula, $v = c\sqrt{rs}$. In like manner Fig. 8 shows, for each observation, the value of v and that of f, the coefficient in the formula for loss of head,

$$H' = f \frac{l}{d} \frac{v^2}{2g} \qquad (11)$$

From an inspection of Figs. 6, 7 and 8 it is evident that the observations do not clearly disclose any law of variation of the coefficients c and f with the velocity of flow. The curves which appear to best represent the distribution of the plotted points are $A\ B$ (Fig. 7) and $A'B'$ (Fig. 8). [It is to be noticed that the value of f from observation No. 3 (0.0549) is so great as to throw the point beyond the limit of Fig. 8.] These curves show, between $v = 1.5$, and $v = 2.5$, a slight decrease in c and increase in f; and from $v = 2.5$ to $v = 4.0$, a slight increase in c and decrease in f. Such a law of variation cannot, however, be regarded as well established by the results shown.

It is apparent that, for values of v less than 4 ft. per second, the values of c and f do not greatly vary with the velocity. The average of all the values of c given in Table No. 3 is 109.1; omitting cases in which v is less than 1 ft. per second, the average is 109.4. The corresponding values of f are 0.0216 and 0.0215.

In addition to the above quantities, it may be well to record the values of n, the coefficient of roughness in Kutter's formula. The above mean value of c gives for n values ranging from about 0.013 to 0.015, the smaller values corresponding to low velocities of flow and the larger to the higher velocities.

Degree of Reliability of Results.

The reliability of the measurement of rate of discharge has already been discussed.

In the determination of loss of head in the steel pipe a difficulty was met which somewhat impaired the reliability of the results. This was the difficulty of making the manometer reservoir and the pipe containing the mercury column perfectly tight under the high pressure existing at the lower manometer station. After the experimenting had continued for several days, it was found that there had been a leakage of mercury sufficient to appreciably affect the manometer readings. As the fluctuations of the surface of the mercury in the reservoir could not be accurately measured, the changes in the quantity of mercury in the manometer introduced errors into the observed values of y (the difference between heights of mercury columns). It has already been pointed out that a constant error in these values is of no consequence; but the leakage of mercury introduced a variable error of uncertain amount. Had the difficulty been foreseen, means

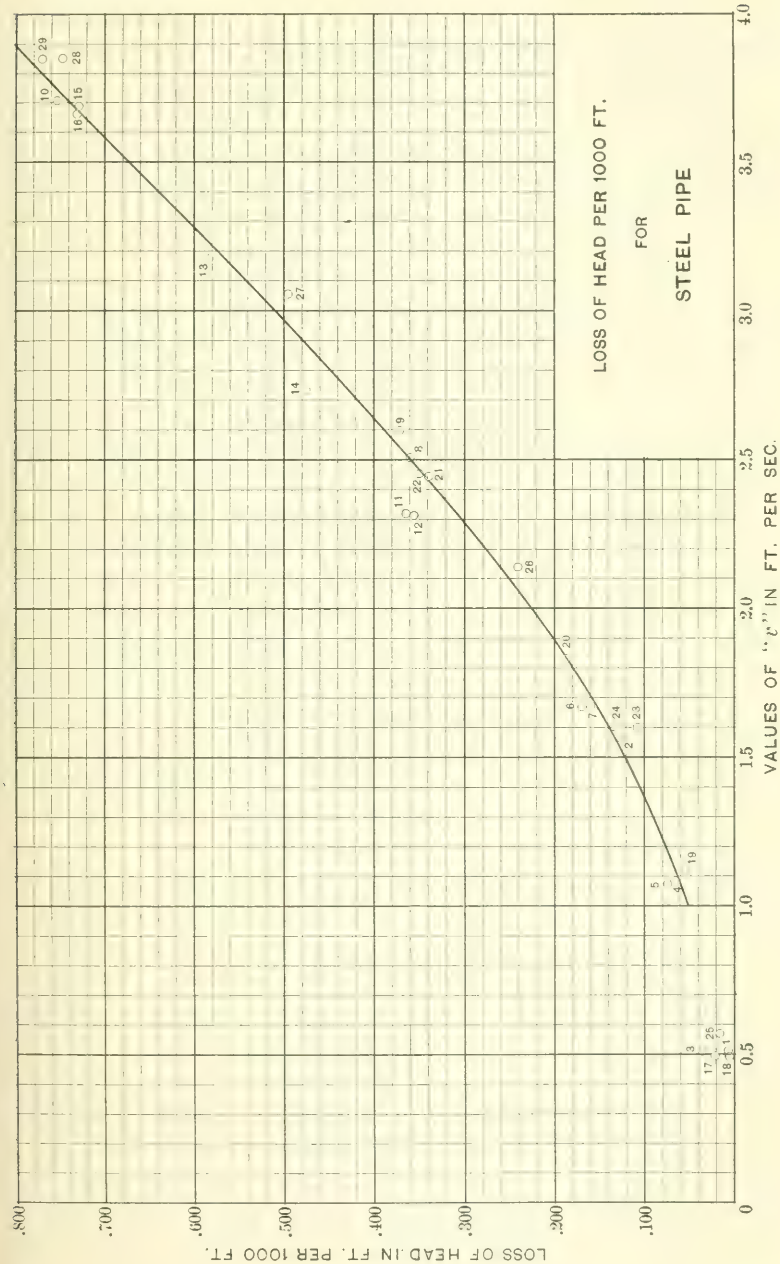

LOSS OF HEAD PER 1000 FT.
FOR
STEEL PIPE
VALUES OF "v" IN FT. PER SEC.
LOSS OF HEAD IN FT. PER 1000 FT.
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
.100
.200
.300
.400
.500
.600
.700
.800

Fig. 6.

would have been provided for reading accurately the position of the mercury surface in the reservoir.

A careful study of the results made it evident that the leakage was very slow, the only case in which a marked change in the static readings was noticed being after the manometer had remained under pressure over night. During the latter portion of the series of experiments, the leakage apparently decreased so much as to be of little importance.

Had it been possible to secure reliable readings of the heights of the manometer columns, under static conditions on each day on which observations were made, the effect of leakage could have been estimated and the errors due to it eliminated. This was not always possible, but a sufficient number of static readings was obtained so that a fair estimate could be made of the values of "static difference of mercury columns" to be used with the different observations. These values, as nearly as could be estimated, are given in column 6 of Table No. 2. The value of the "static difference" shows an increase at observation No. 6, and again at No. 13. This was due to the addition of mercury to the lower manometer.

It is believed that the greater part of the irregularity observed in the values of loss of head per thousand feet, and in the values of c and f, is due to the leakage of mercury from the lower manometer. A further possible cause of error, especially in the earlier observations, may have been the presence of air in the pipes leading from the main pipe to the manometers. In the case of the lower manometer, the connecting pipe was necessarily of considerable length, since the only practicable location for the manometer was in the power-house, and careful manipulation was needed to keep so long a pipe wholly free from air. This was probably accomplished more successfully during the later observations than during the earlier ones.

The possible errors due to changes of temperature of the mercury columns, and the method of correcting for such changes, have already been discussed. As a further possible source of error, changes of temperature of the water in the main pipe may be mentioned. It was not practicable to observe the temperature of the water within the pipe during the time of experimenting. The best that could be done was to take the temperature of the water in the waste flume below the power-house. No complete series of temperature observations was

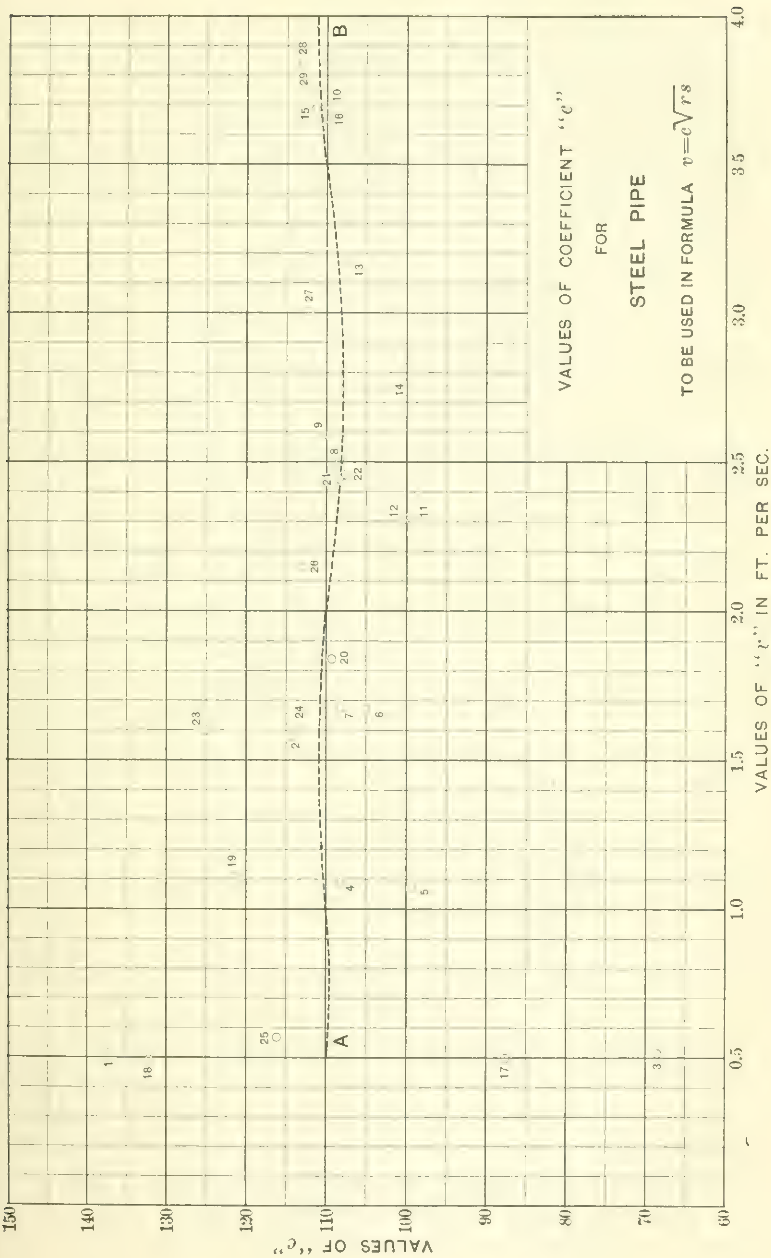

VALUES OF COEFFICIENT "c"
FOR
STEEL PIPE
TO BE USED IN FORMULA $v=c\sqrt{rs}$
VALUES OF "v" IN FT. PER SEC.
VALUES OF "c"
A
B
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
60
70
80
90
100
110
120
130
140
150

Fig. 7.

taken, but frequent observations taken during the time from August 4th to August 11th showed a variation of about 2° Cent., the range being from 16 to 18 degrees. The coefficient of expansion of water at this temperature is about 0.000174, nearly as great as that of mercury. A variation of the temperature of the water from 16° to 18° Cent. would change the value of the difference of pressure at the two manometer stations by about 0.1 ft. of water, and would appreciably affect the manometer readings, though its importance would not be great, except for small values of the loss of head measured.

Although the plotted results (Figs. 6, 7 and 8) indicate the presence of errors sufficiently large to be of importance in comparison with the quantities measured, yet it will be seen that the absolute values of the errors affecting the pressure measurements are not great. It was thought that the method adopted for measuring loss of head should give results reliable to within 0.1 ft. in absolute value, and it is believed that, even with the difficulties mentioned, the results actually reached possess a degree of reliability as high as the standard set. Thus, the greatest discrepancy in the whole series is shown by observations Nos. 1 and 3. The mean velocity has the same value in the two cases, while No. 1 gives the least and No. 3 the greatest of all the values found for the coefficient c. The values of the total loss of head differ by only 0.122 ft. (column 8, Table No. 2), and the actual error in each value is doubtless within the 0.1 ft. mentioned as the standard of accuracy expected.

It may be noted further, that, if the value of c is regarded as constant, and its value obtained by averaging all the values found, excluding cases in which v is less than 1 ft. per second, the probable error of this mean value is about 0.9.

Comparison of Known Data Regarding the Discharge of Riveted Pipes.

So far as known to the authors, no experiments upon the discharge of new riveted pipes of diameter as great as 6 ft. have been made. Experiments upon riveted pipes of smaller sizes have been too few to furnish a basis for establishing the law of variation of the coefficient c with the velocity, diameter and other conditions. The resistance to flow offered by the pipe doubtless depends upon various elements—thickness of plates, length of sections, frequency of rivets and size of

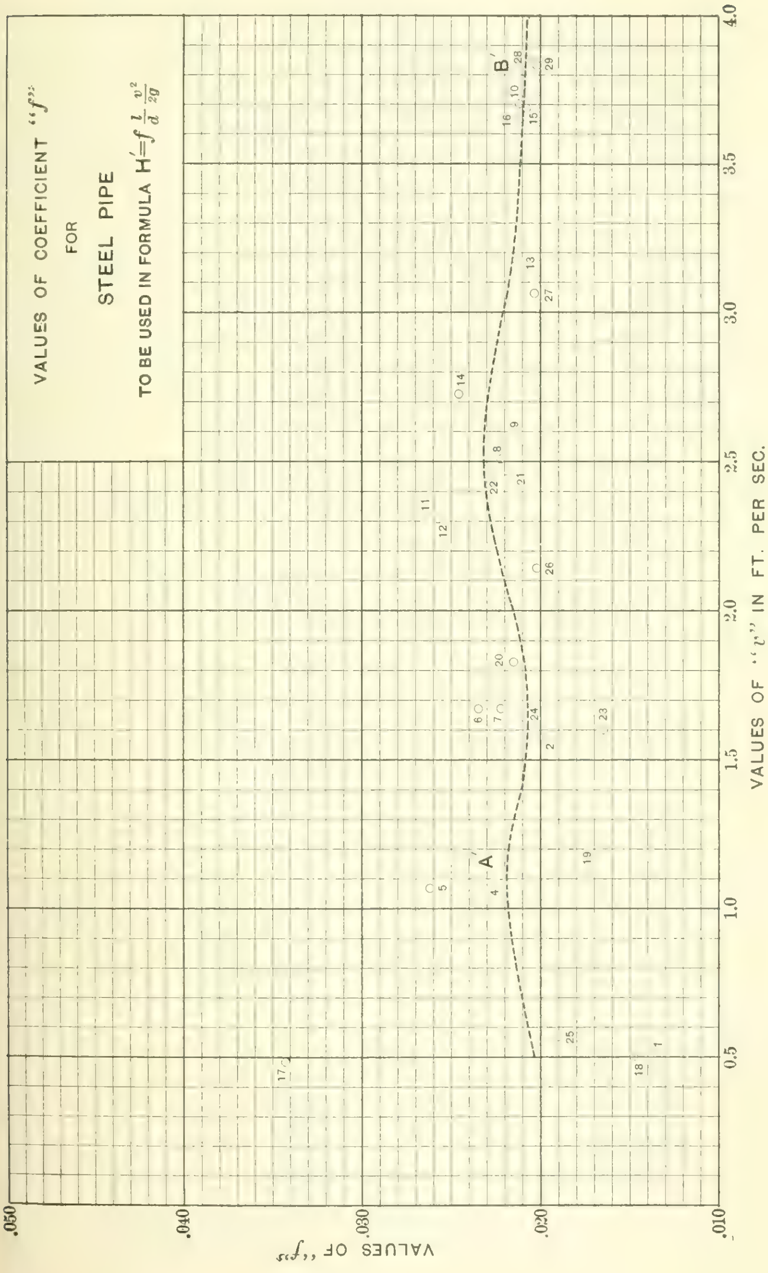

VALUES OF COEFFICIENT "f"
FOR
STEEL PIPE
TO BE USED IN FORMULA $H'=f\frac{l}{d}\frac{v^2}{2g}$
A'
B'
VALUES OF "v" IN FT. PER SEC.
VALUES OF "f"
.050
.040
.030
.020
.010
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

FIG. 8.

TABLE No. 4.—EXPERIMENTS ON RIVETED PIPES,

No.	1	2	3	4	5	6	7	8	9	10	11	12
Age.	5 yrs.	New.	5 yrs.	5 yrs.	New.	1 yr.	14 yrs.	14 yrs.	5 yrs.	New.	New.	14 yrs.
Diam. (ins.)...	11	11.25	13	15	16	17	24	24	26	33	35	36
Vel.	Coefficient c.											
0.5												
1.0												
1.3		101.3										
1.48												80.3
1.5												
2.0												
2.44												
2.5												
2.78		114.0										
3.0												
3.23												
3.27												
3.32							75.8	78.5				
3.5												
3.52											126.8	
3.87			121.6									
3.90												
3.96										123.2		
4.0												
4.38				111.6								
4.5												
4.58					112.3							
4.60			109.4									
4.71	107.1											
4.90		122.5										
4.93												
5.0												
5.5												
6.0												
6.09	110.6											
6.67		126.5										
6.84				117.8								
6.93	111.5											
6.96			113.4									
7.31				119.1								
8.46				119.0								
8.65			113.0									
8.66	113.4											
8.85		128.1										
10.02	115.5											
10.52		129.9										
10.59				121.6								
10.71			114.4									
12.09				121.3								
12.60									134.1			
20.14						131.1						
n	.010	.010	.011	.011	.011	.010	.017	.016	.010	.011	.011	.017

WITH DIAMETERS RANGING FROM 11 INS. TO 103 INS.

No.	13	13a	14	15	16	17	18	19	19a	19b	20	21	22
Age.	New.	4 yrs.	New.	New.	New.	New.	New.	New.	4 yrs.	4 yrs.	New.	New.	5 yrs.
Diam. (ins.)	36	36	38	38	42	42	42	48	48	48	48	72	103
Vel.	Coefficient c.												
0.5												110	126.5
1.0	86.0					96.0	101.0	101.2	78.0	97.2	97.1	110	116.6
1.3													
1.48													
1.5	90.6					103.0	102.8	105.4	84.6	100.8	98.7	111	112.7
2.0	95.2					107.9	104.3	108.8	89.6	103.3	100.3	110	110.3
2.44					115.9								
2.5	99.4					111.0	105.5	111.2	92.4	104.9	101.6	108	108.8
2.78													
3.0	103.3					112.6	106.4	112.8	93.0	105.3	102.2	108	107.7
3.23			114.0										
3.27			116.6										
3.32													
3.5	107.0					113.0	107.2	113.4	93.2	104.8	103.6	110	106.9
3.52													
3.87													
3.90				109.2									
3.96													
4.0	110.6					112.8	107.8	113.2	94.0	104.0	104.2	111	106.2
4.38													
4.5	114.0					111.8	108.2	112.4	94.2	103.7	104.7		105.6
4.58													
4.60													
4.71													
4.90													
4.93		106.3											
5.0	117.2					110.8	108.4	112.0	94.4	103.7	105.1		
5.5	120.4					110.2	108.5	111.7	94.7	103.7	105.2		
6.0	123.6					110.0	108.5	111.6	94.9	103.7	105.2		
6.09													
6.67													
6.84													
6.93													
6.96													
7.31													
8.46													
8.65													
8.66													
8.85													
10.02													
10.52													
10.59													
10.71													
12.09													
12.60													
20.14													
n	.014	.013	.013	.013	.013	.013	.013	.013	.016	.014	.014	.014	.015

rivet-heads, nature of interior coating, and the number, frequency and sharpness of bends or elbows. These various conditions differ so much in the pipes hitherto experimented upon that hardly any two sets of results are directly comparable. The available data, so far as known to the authors, for pipes not less than 11 ins. in diameter, are shown in Table No. 4. The values of c are arranged according to values of the diameter, and also of the mean velocity of flow. At the bottom of each vertical column is given the value of n, the Kutter coefficient of roughness, as computed from the corresponding series of experiments. These values have not been computed with great precision, being taken from Kutter's diagram; and in case of experiments covering a considerable range of velocities, the recorded value of n is intermediate between the extreme values given by the experiments. Additional particulars regarding the different series of experiments from which the tabular results are deduced will now be given.

1, 3 and 4. Experiments by Hamilton Smith, M. Am. Soc. C. E., at North Bloomfield, Cal., in October, 1876. Pipes of sheet iron, single riveted, made in lengths of about 20 ft., "stove-pipe" or taper joints. Coated by immersion in boiling asphaltum and coal tar. In use 5 years, but interior surfaces worn smooth. Length in every case about 700 ft. Discharge measured by weirs. Loss of head by leveling between surface of supply reservoir and outflowing stream, correcting for velocity head. Smith's "Hydraulics," p. 302.

2. Experiments by Darcy, 1849–51. New sheet-iron pipe, coated with bitumen; screw joints; length, 365.5 ft. Discharge determined by measurement in tanks; loss of head (probably) either by water piezometers or mercury gauges. The tabulated results, in English units, are taken from Smith's "Hydraulics," p. 226.

5. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Ore., Water-Works. Steel pipe, cylinder joints; 16 ins. is diameter of small sections; asphalt coating. Discharge measured by rise of reservoir surface in test of 18 hours; loss of head by open stand-pipes. The measurement of loss of head for steel pipe was indirect; the total loss in the combined lengths of 19 130 ft. of stave pipe and 16 416 ft. of steel pipe was measured, and at the same time the loss in 4 188 ft. of stave pipe alone. *Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 21.

6. Experiment by Hamilton Smith at Texas Creek, Cal. Pipe of sheet iron, made in lengths of 20 ft.; heavy coating of asphaltum and coal tar. Length, 4 439 ft. Discharge measured by weir, 5.5 ft. long; loss of head by leveling between surfaces of supply and discharge tanks. Smith's "Hydraulics," p. 311.

7. Experiment by George W. Rafter, M. Am. Soc. C. E., on old conduit of Rochester, N. Y., Water Works, July and August, 1890. Cylinder joints. Length of experimental section, 1 901 ft. Discharge determined by rise of reservoir surface; loss of head by water piezometers, *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 20.

The tabulated results are based upon a discharge of 6 742 000 galls. per day. Four gaugings of this conduit, made when it was new (in 1876), gave as an average 8 704 000 galls. per day.* Assuming equal ratio of decrease of capacity to have taken place in all parts of the conduit, the original gaugings would give about 97 as the value of c (for $v = 4.25$) when the pipe was new.

8. Same experiment as No. 7, but a different portion of the conduit. Length, 10 541 ft. Loss of head measured by Bourdon gauges.

9. Experiment by Hamilton Smith on "Humbug" pipe of North Bloomfield (Cal.) Mining Company. Pipe of sheet iron, coated with asphaltum and coal tar. Laid in 1868; experiment made in 1873, the surface being then quite smooth. Measurement of discharge not regarded as very exact. Loss of head measured by leveling between surfaces of supply and discharge reservoirs. Length, 1 194 ft. Smith's "Hydraulics," p. 309.

10. Experiment by Isaac W. Smith, M. Am. Soc. C. E., on conduit of Portland (Ore.) Water-Works. Cylinder joints; asphalt coating. Observations made on February 27th, 1896. Discharge (probably) measured by weir; loss of head by water piezometers. Length, 34 176 ft. *Trans. Am. Soc. C. E.*, Vol. xxvi, p. 203.

11. Same experiment as No. 10; another portion of conduit. Length, 39 809 ft.

12. Same experiment as Nos. 7 and 8; another portion of conduit. Loss of head measured by water piezometers. Length, 50 819 ft.

The mean of the above-mentioned four measurements made in 1876 would give, for this section of pipe, a value of c of about 103 for $v = 1.9$; assuming equal ratio of decrease of capacity for all parts of the conduit.

13. Experiments by Clemens Herschel, M. Am. Soc. C. E., on conduit of the East Jersey Water Company, from Belleville to South Orange Avenue, 1892. Cylinder joints, asphalt coating. Discharge measured by "imperfect weir;" loss of head by Bourdon gauges. Length, 25 000 ft. Herschel, "115 Experiments on the Carrying Capacity of Large, Riveted, Metal Conduits," pp. 28, 52.

13*a*. Same conduit as No. 13; experiment by Herschel in 1896. Discharge measured by Venturi meter; loss of head by Bourdon gauges. Length, 24 720 ft. "115 Experiments," pp. 28, 52.

14. Experiments by Emil Kuichling, M. Am. Soc. C. E., on new conduit of Rochester Water-Works between Hemlock Lake and Rush

* "Annual Reports Executive Board, Rochester, N. Y." 1894–95, p. 169.

Reservoir, October and December, 1895. Cylinder joints, inner sections 38 ins. in diameter; part coated with asphalt, part with Sabin's japan. Length, 91 641 ft. Discharge measured by rise of reservoir surface; loss of head by reading mercury gauge at lower end and observing level of water in feeding chamber at upper end. "Annual Reports Executive Board, Rochester, N. Y.," 1894–95, p. 344.

A portion of this conduit (from West Bloomfield to Rush Reservoir, 34 468 ft.) was tested again in June and October, 1896, giving $c = 90.04$ as mean of 22 observations. Methods of measurement and range of velocities not stated; "total fall in hydraulic grade line" ranged from 1.08 ft. to 6.95 ft. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1896, p. 45.

15. Experiments by Emil Kuichling on new Rochester conduit between Rush Reservoir and Mt. Hope Reservoir, October and November, 1895. Construction same as No. 14; coated with Sabin's japan. The tabulated results are the mean of three gaugings agreeing closely among themselves. "Ann. Rep. Exec. Board, Rochester, N. Y.," 1894–95, p. 349.

16. Same experiment as Nos. 10 and 11; another portion of conduit; construction the same. Length, 50 965 feet.

17. Experiments by Clemens Herschel on Kearney Extension of East Jersey Water Company. Taper joints, coating "unusually smooth." Conduit put in use January 10th, 1896; experiments made January 21st–30th, 1896 (except one November 18th, 1896). Length, 5 574 ft. Greatest velocity recorded, 4.26. Discharge measured by Venturi meter: loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

18. Experiments by Clemens Herschel on conduit No. 2 of East Jersey Water Company, below Pompton Notch. Conduit put in use September 30th, 1896; experiments made in September and October, 1896. Length from 49 833 to 81 139 ft. Highest recorded velocity, 5.41. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 29, 53.

19. Experiments by Clemens Herschel on Conduit No. 1 of East Jersey Water Company, 1892. Observations made on various portions of whole line, lengths ranging from 24 630 to 74 396 ft. Cylinder joints, asphalt coating. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 26, 27, 52.

19*a*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to section above Pompton Notch. Length ranging from 10 507 to 24 630 ft. Highest recorded velocity, 4.63. Discharge and loss of head measured as in No. 19. "115 Experiments," pp. 27, 52.

19*b*. Experiments by Clemens Herschel in 1896. Same conduit as No. 19, but experiments limited to portion below Pompton Notch.

Length ranging from 26 610 to 83 000 ft. Highest velocity recorded, 6.06. "115 Experiments," pp. 27, 52.

20. Experiments by Clemens Herschel in 1896 on Conduit No. 2 of East Jersey Water Company, above Pompton Notch. Taper joints; length, 24 648 ft. Velocity ranged from 2.99 to 4.69. Discharge measured by Venturi meter; loss of head by Bourdon gauges. "115 Experiments," pp. 28, 53.

21. Experiments on conduit of the Pioneer Electric Power Company, Ogden, Utah, August, 1897. Butt joint, asphalt coating. Length, 4 367 ft. and 4 427 ft. Velocity ranged from 0.51 to 3.85. Discharge measured by Venturi meters; loss of head by mercury gauges.

22. Experiments by Clemens Herschel in 1887, on feeding trunk of Holyoke testing flume. Diameter nominally 9 ft.; actual diameter of mean section, 8.58 ft. Cylinder joints; paint coating worn off; rather rusty. Length, 152.88 ft. Discharge measured by "accurate weir"; loss of head by water piezometers (surfaces of water in still boxes determined by hook gauges). *Trans. Am. Soc. C. E.*, pp. 246, 247. "115 Experiments," pp. 29, 54.

It will be noticed that in the case of the authors' results (column 21, Table No. 4), the values of c are given only to the nearest unit, while in every other case the first decimal figure is given. It is not to be inferred from this that these results are regarded as less precise in character than the others. In the opinion of the authors no experiments have been made of such a character as to warrant the use of more than three significant figures in the values of c. In certain cases experimenters have recorded values of c to the sixth significant figure. Such an appearance of precision would be warranted if the results were reliable to within one-thousandth of 1 per cent.

Very little in the way of general conclusions as to the carrying capacity of large riveted conduits can be deduced from the data above recorded. It is evident that the capacity of such conduits is less than was formerly supposed. The experiments of Hamilton Smith on pipes of the smaller sizes (Nos. 1, 3, 4, 6 and 9, Table No. 4) had been supposed to warrant the conclusion that riveted pipes had nearly or quite as great carrying capacity as smooth cast-iron pipes. For such pipes it has been supposed that the value of c increases with the velocity and also with the diameter; the table and curves given by Hamilton Smith*

* "Hydraulics," p. 271 and Pl. XIV. From an inspection of Pl. XIV. it appears that the curves showing the relation between c and v for diameters greater than 1 ft. are based on very meager data, and that the curves as drawn do not conform to the few experimental points recorded. It seems to be arbitrarily assumed that the law of variation of c for large diameters must show a general agreement with the law determined by experiment for small diameters.

being regarded as safe guides in the selection of coefficients. The results recorded in Table No. 4 under numbers 13 to 22 indicate that for large riveted pipes Smith's values are much too great. It is also to be noticed that these results show little or no variation of c with the diameter, and no decided variation with the velocity. It would appear that, for conduits of 36 ins. diameter or over, it is unsafe to count on a value of c greater than 110 for new pipes; if there are frequent or sharp bends the coefficient will doubtless be still less. Moreover, in case of long conduits under ordinary conditions of use, it is to be expected that the capacity will suffer considerable diminution with age.

It should be said that at the time the Ogden conduit was designed, the views of the engineering profession as to the carrying capacity of large riveted conduits fully warranted the use of a coefficient of 120. The construction of the conduit was such that its "smoothness" would certainly appear to compare favorably with any previously made. The butt joints give an interior surface without breaks, there being no interior butt straps at the round joints, while the longitudinal straps are continuous throughout the length of the conduit. As regards smoothness, this construction would appear to have a distinct advantage over either "cylinder" or "taper" joints.

A remark may be made regarding the applicability of Kutter's formula. From Table No. 4 it appears that, for diameters less than 36 ins., and for pipe which is either new or in such condition as to be probably as smooth as when new, the value of n shows little variation in the different experiments, being 0.010 or 0.011 in every case. For larger sizes there is an increase of n with the diameter. This does not appear to be accounted for by a greater roughness of the larger pipes experimented on, but is rather to be attributed to the imperfection of the Kutter formula. It is further true that experiments on the same pipe with different velocities (and consequently different values of the slope s) give different values of n.

III.—Experiments on Wooden Pipe.

Description of Pipe Line.

The plan, profile, dimensions and construction of the entire pipe line may be found in the paper, already referred to.* The plan and

* *Transactions*, Vol. xxxviii, p. 246.

profile of the portion of the wooden pipe upon which the experiments were made are shown in Fig. 9, *C* being the lower and *D* the upper pressure station. The diameter of the pipe is 72.5 ins.

Record of Observations.

(1) *Example of Plotted Results.*—Reference has already been made to Fig. 5, showing graphically the record of the two manometers and of the Venturi meter difference-gauge for a part of the run made on August 16th on the wooden pipe. Four observations of the series made on the wooden pipe were taken at times of falling pressure, giving manometer records similar to that shown in Fig. 4, although the latter record belongs to an experiment on the steel pipe.

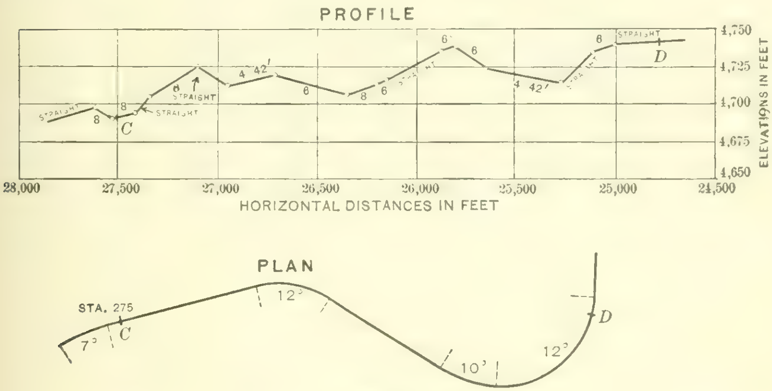


Fig. 9.

(2) *Tabulation of Reduced Observations.*—From the experiments made upon the wooden pipe, 22 "observations" resulted. The reduced data of these observations are given in Tables Nos. 5, 6 and 7, of which no explanation is needed, since they are in all respects similar to the tables showing data of observations on the steel pipe (Tables Nos. 1, 2 and 3).

The date and period of each observation are given in Table No. 5. It will be noticed that observations 40, 44, 45 and 46 were made under conditions of falling pressure.

(3) *General Results.*—Table No. 7 shows the observations arranged in order of the values of the mean velocity of flow, three sets of results

TABLE No. 5.—WOODEN PIPE. DISCHARGE OBSERVATIONS.

1	2	3	4	5	6	7	8	9
			NORTH VENTURI METER.		SOUTH VENTURI METER.			
No.	Date.	Time.	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Reading of difference-gauge. (Feet.)	Rate of discharge. (Cubic feet per second.)	Rate of discharge of main pipe. (Cubic feet per second.)	Mean velocity of flow in main pipe. (Feet per second.)
	1897.							
30....	Aug. 15.	8.30— 9.00	....	....	0.054	23.4	23.4	.816
31....	" 15.	9.28— 9.45	....	....	.178	42.5	42.5	1.482
32....	" 15.	10.14—10.30	....	....	.376	61.3	61.3	2.138
33....	" 15.	10.50—11.15	.028	16.7	.366	60.4	77.1	2.689
34....	" 15.	11.32—12.00	.150	38.9	.363	60.1	99.0	3.453
35....	" 16.	8.01— 8.30	....	....	.024	15.3	15.3	.534
36....	" 16.	8.45— 8.52	....	....	.122	35.2	35.2	1.228
37....	" 16.	8.54— 9.15	....	....	.125	35.6	35.6	1.242
38....	" 16.	9.50—10.05	....	....	.406	63.6	63.6	2.218
39....	" 16.	10.26—10.45	.028	16.7	.400	63.1	79.8	2.783
40....	" 16.	11.00—11.30	.156	39.6	.395	62.7	102.3	3.568
41....	" 17.	8.30— 9.00	....	....	.036	19.1	19.1	.666
42....	" 17.	9.28— 9.45	....	....	.145	38.2	38.2	1.332
43....	" 17.	10.16—10.30	....	....	.321	56.7	56.7	1.978
44....	" 17.	10.48—11.15	.174	42.1	.390	62.4	104.5	3.645
45....	" 17.	11.30—11.41	.174	42.1	.383	61.9	104.0	3.627
46....	" 17.	11.50—12.00	.174	42.1	.380	61.6	103.7	3.617
47....	" 18.	8.30— 9.00	....	....	.025	15.6	15.6	.544
48....	" 18.	9.15— 9.45	....	....	.120	34.9	34.9	1.217
49....	" 18.	10.25—10.30	....	....	.288	53.8	53.8	1.876
50....	" 18.	10.56—11.15	.032	18.1	.297	54.6	72.7	2.536
51....	" 18.	11.36—12.00	.130	36.2	.292	54.1	90.3	3.149

being given as in the case of the steel pipe already discussed. The quantities in this table (values of c, f and loss of head per 1 000 ft.) are also represented graphically in Figs. 10, 11 and 12.

These results differ from those found for the steel pipe in showing a well-marked variation of c and f with the velocity of flow.

Fig. 11 shows that c increases with the velocity in somewhat the way usually assumed for smooth cast-iron pipes. The highest velocity obtained in the experiments was about 3.64 ft. per second. The highest velocity, not accompanied by a fall of pressure, was about 3.45 ft. per second (observation 34). The four observations giving highest velocities are somewhat discrepant, due, probably, to the difficulty of determining accurately the difference between the heights of manometer columns during periods of decreasing pressure.

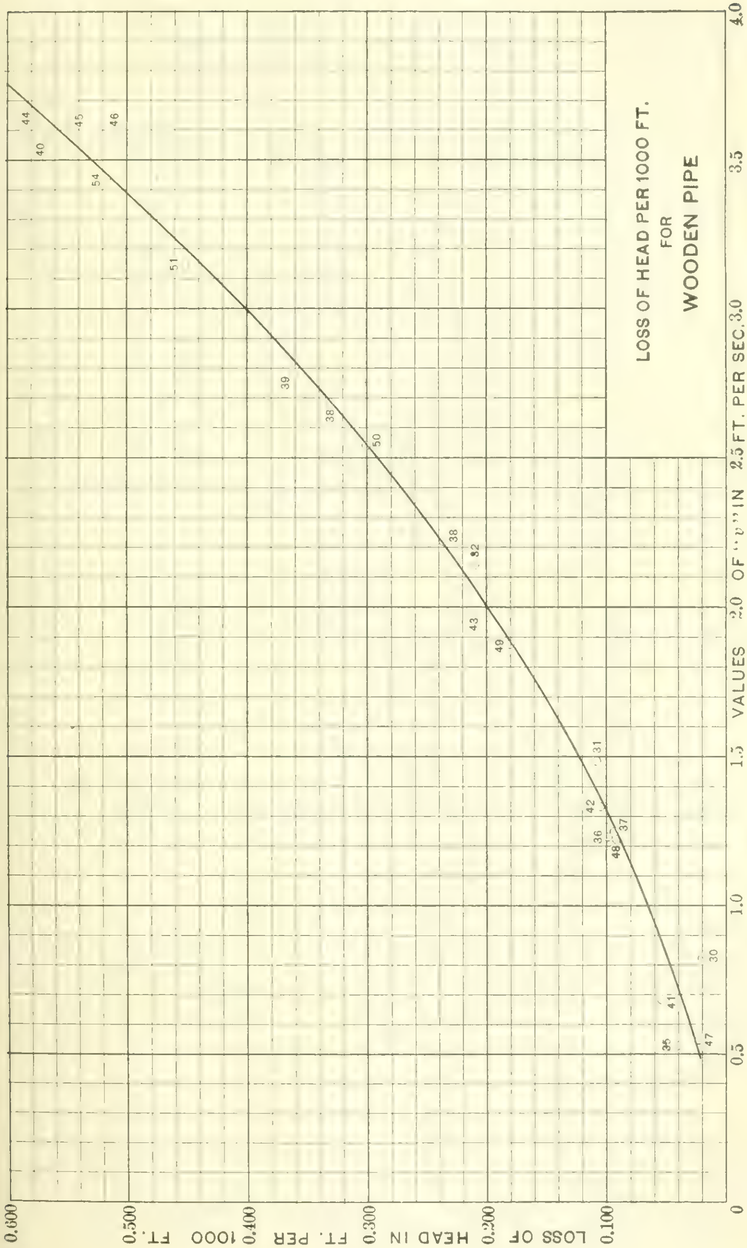

LOSS OF HEAD PER 1000 FT.
FOR
WOODEN PIPE
VALUES OF "v" IN FT. PER SEC.
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
LOSS OF HEAD IN FT. PER 1000 FT.
0.100
0.200
0.300
0.400
0.500
0.600

Fig. 10.

TABLE No. 6.—WOODEN PIPE.—MANOMETER OBSERVATIONS.

1	2	3	4	5	6	7	8	9
No.	Length of pipe. (Feet.)	Heights of Manometer Columns Reduced to 0° Cent. (Feet.) Upper.	Lower.	Difference.	Static difference of mercury columns reduced to 0° Cent. (Feet.)	Loss of Head between Manometers. Mercury. (Feet.)	Water. (Feet.)	Loss of head per 1 000 ft. in feet of water.
30..	2 710	3.474	7.364	3.890	3.894	.004	.054	.0201
31..	2 710	3.384	7.256	3.872	3.894	.022	.299	.111
32..	2 710	3.151	7.003	3.852	3.894	.042	.571	.211
33..	2 710	2.882	6.710	3.828	3.884	.066	.898	.331
34..	2 710	2.434	6.225	3.791	3.894	.103	1.401	.517
35..	2 710	3.496	7.382	3.886	3.894	.008	.109	.0401
36..	2 710	3.446	7.320	3.874	3.894	.020	.272	.1003
37..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0952
38..	2 710	3.116	6.963	3.847	3.894	.047	.639	.235
39..	2 710	2.833	6.655	3.822	3.884	.072	.979	.361
40..	2 710	Dropping	Dropping	3.781	3.894	.113	1.537	.566
41..	2 710	3.486	7.369	3.883	3.894	.011	.150	.0552
42..	2 710	3.417	7.290	3.873	3.894	.021	.286	.106
43..	2 710	3.211	7.063	3.852	3.894	.042	.571	.211
44..	2 710	Dropping	Dropping	3.779	3.894	.115	1.564	.576
45..	2 710	"	"	3.785	3.894	.109	1.482	.547
46..	2 710	"	"	3.794	3.894	.100	1.360	.501
47..	2 710	3.492	7.381	3.889	3.894	.005	.068	.0251
48..	2 710	3.443	7.318	3.875	3.894	.019	.258	.0953
49..	2 710	3.252	7.110	3.858	3.894	.036	.490	.181
50..	2 710	2.985	6.819	3.834	3.894	.060	.816	.300
51..	2 710	2.628	6.432	3.604	3.894	.090	1.224	.452
	....	3.518	7.412	3.894	3.894	.000	.000	.000

Reliability of Results.

Excluding observation 30, the experiments on the wooden pipe are more accordant than those on the steel pipe. This is, doubtless, due to the fact that the most important source of error in the steel pipe experiments was absent in the later series.

The greatest pressure to which either manometer was subjected in this series was due to a head of about 101 ft., as against 462 ft. in the former series. Under this lower pressure no difficulty was experienced in securing tight joints, and leakage of mercury did not occur. The long connecting pipe between the manometer and the main pipe (which was necessary at the lower end of the steel pipe) was also avoided, thus diminishing the difficulty of eliminating air. The temperature conditions were, on the whole, as favorable in the second series as in the first; for although neither manometer was furnished with a water jacket, the upper one was very completely sheltered from temperature changes, while the lower was not seriously exposed. The fact that

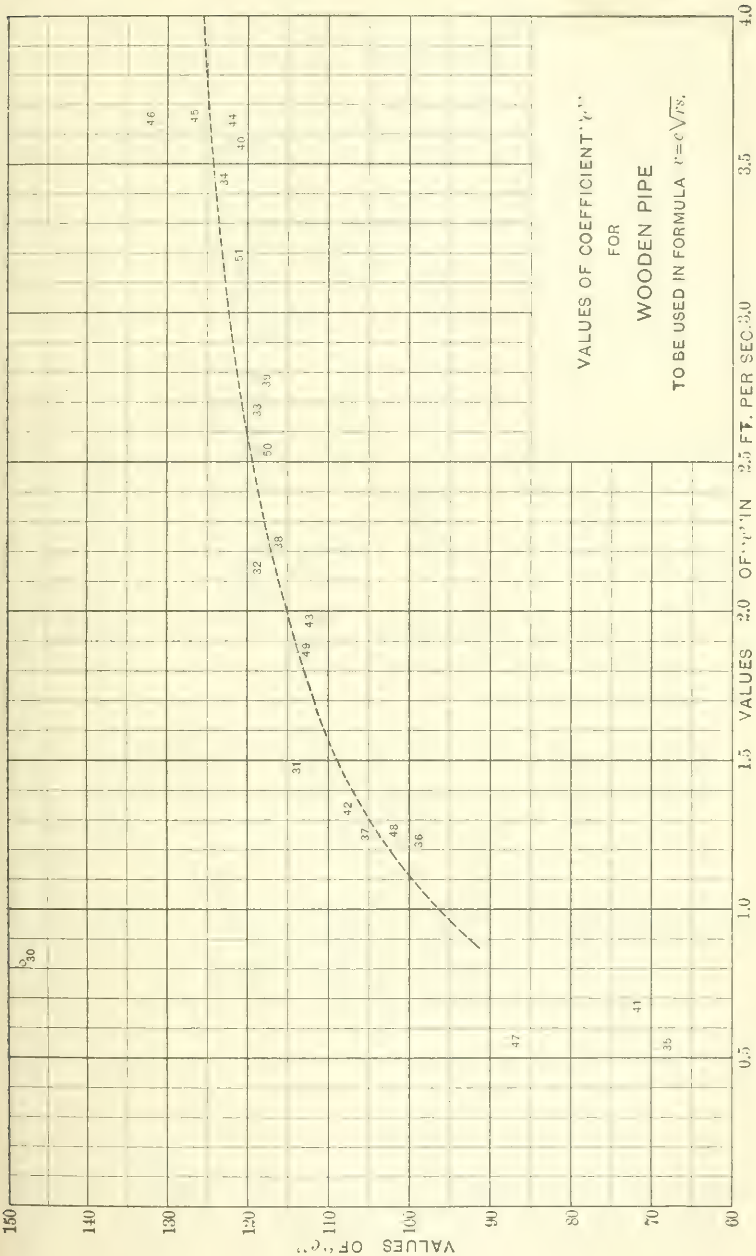

VALUES OF COEFFICIENT "c"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $v=c\sqrt{rs}$.
VALUES OF "v" IN FT. PER SEC.
VALUES OF "c"
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
60
70
80
90
100
110
120
130
140
150

Fig. 11.

TABLE No. 7.—Wooden Pipe. General Results.

1	2	3	4	5
No.	Mean velocity in main pipe. (Feet per second.)	Loss of head per 1 000 ft. (Feet.)	Value of c in formula. $v = c\sqrt{rs}$	Value of f in formula $H' = f\frac{l}{d}\frac{v^2}{2g}$
35........	.534	.0401	69	.0547
47........	.544	.0251	88	.0262
41........	.666	.0552	73	.0483
30........	.816	.0201	148	.0117
48........	1.217	.0953	101	.0250
36........	1.228	.1003	100	.0259
37........	1.242	.0952	104	.0241
42........	1.332	.106	106	.0230
31........	1.482	.111	115	.0195
49........	1.876	.181	114	.0199
43........	1.978	.211	111	.0209
32........	2.138	.211	120	.0179
38........	2.218	.235	117	.0186
50........	2.536	.300	119	.0182
33........	2.689	.331	120	.0178
39........	2.783	.361	119	.0181
51........	3.149	.452	121	.0177
34........	3.453	.517	124	.0168
40........	3.568	.566	122	.0173
46........	3.617	.501	131	.0149
45........	3.627	.547	126	.0161
44........	3.645	.576	123	.0168

both columns were much shorter also rendered the temperature correction less important.

Known Data Regarding the Capacity of Wooden Pipes.

Besides the foregoing results, the authors know of only the following experimental data regarding the carrying capacity of wooden stave pipe.

1. Experiment by Arthur L. Adams, M. Am. Soc. C. E., on conduit of Astoria, Oregon, Water-Works. This is the same experiment as that already described in connection with the discussion of steel pipes (No. 5, Table No. 4). The loss of head in 4 188 ft. of 18-in. stave pipe was measured by means of open stand-pipes. The discharge was determined by measuring the rise of the reservoir surface in a test of eighteen hours. The value of c was 132.9 for $v = 3.605$. The Kutter coefficient of roughness was about 0.010.—*Trans. Am. Soc. C. E.*, Vol. xxxvi, p. 26.

2. In an account of the 30-in. stave pipe of the Denver water-works by James D. Schuyler, M. Am. Soc. C. E., it is stated that gaugings were made by measuring the rise of the reservoir surface in a known time, and also by measurement of the velocity by current meter. As a result it is said that in "applying Kutter's formula to wood pipe, as

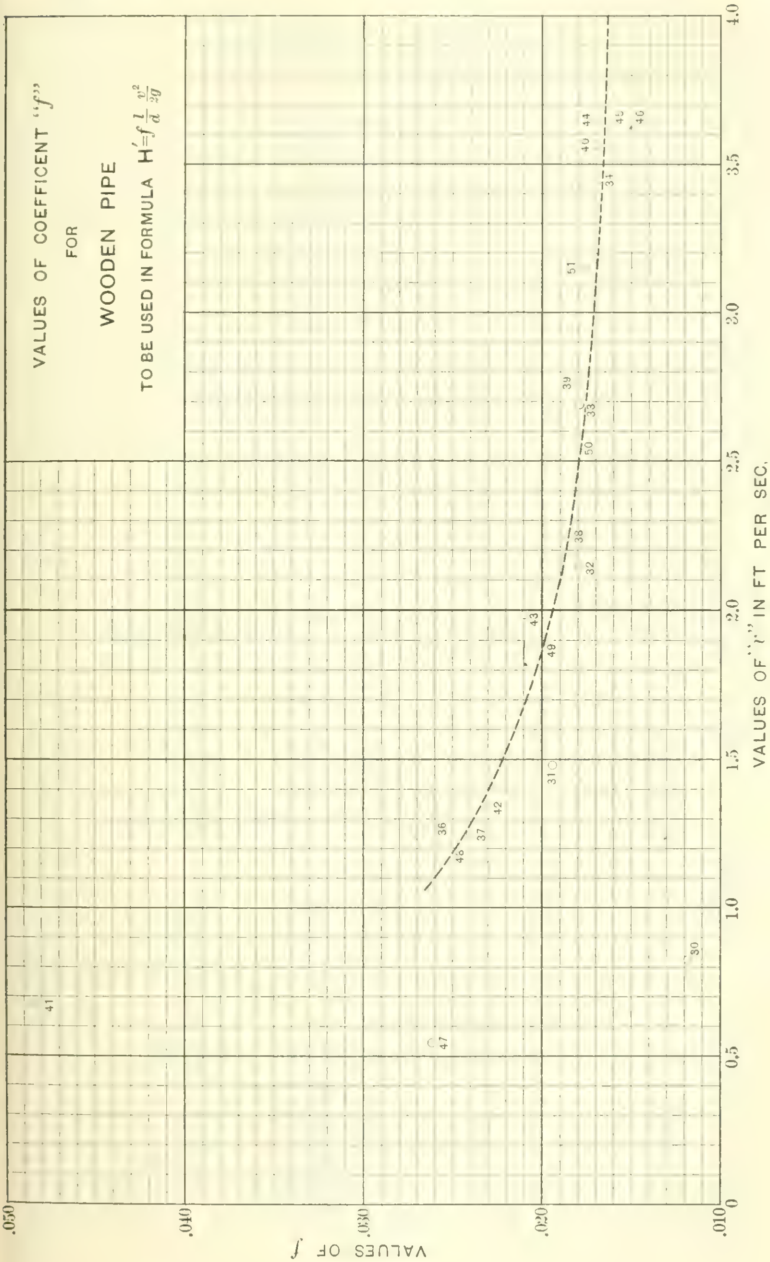

VALUES OF COEFFICENT "f"
FOR
WOODEN PIPE
TO BE USED IN FORMULA $H' = f \frac{l}{d} \frac{v^2}{2g}$
VALUES OF "v" IN FT PER SEC.
VALUES OF f
.050
.040
.030
.020
.010
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0

Fig. 12.

low a coefficient n as 0.0096 can be used." This value of n would give values of c probably between 140 and 150, depending upon the hydraulic slope, which is not given. *Trans. Am. Soc. C. E.*, Vol. xxxi, p. 144.

3. Fred. B. Gutelius, C. E., states that in the design of the 24-in. stave pipe of the Butte City (Montana) Water Company, the value of n was taken as 0.010, and that the correctness of this value was justified by a test. The particulars of the test are not given, but this value of n would give values of c probably between 120 and 130, depending upon the hydraulic slope. *Journal of the Association of Engineering Societies*, Vol. xii, p. 219.

According to Hamilton Smith,* for circular pipe having "quite smooth interior surfaces, and no sharp bends," the value of c increases with the diameter and also with the velocity. Following are the values given by him for diameter 6 ft. and for velocities up to 4 ft. per second, with the corresponding values derived from the experiments on the Ogden pipe by the authors.

v	c	
	Smith's table.	Ogden experiments.
1	131.8	97
2	138.0	115
3	142.3	122
4	145.5	126

It will be seen that, while the two series agree in showing an increase of c with the velocity, the experimental values are decidedly lower than those given in Smith's table.

In this connection it may be noted that the experiments described by Desmond FitzGerald, M. Am. Soc. C. E.,† on clean cast-iron pipe 48 ins. in diameter, show a fair agreement with Smith's tables as regards the general law connecting v and c, but give values of c higher than Smith's for velocities less than 7 ft. per second. On the other hand, 48-in. pipe, with surface much tuberculated, showed a very different series of values of c; there being little variation of c with v except for low velocities, and the variation being a decrease of c with increasing velocity. The values of c for tuberculated pipe were not

* "Hydraulics," p. 271.

† *Trans. Am. Soc. C. E.*, vol. xxxv., p. 241.

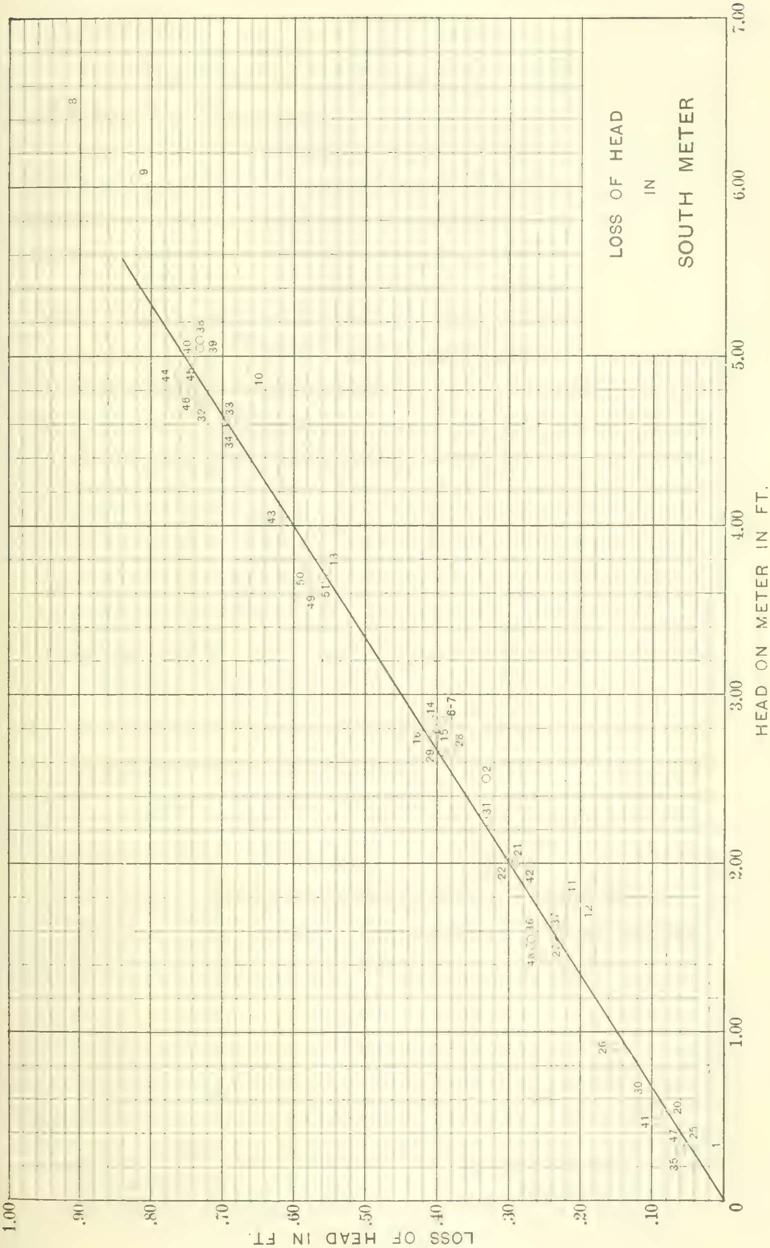

LOSS OF HEAD
IN
SOUTH METER
HEAD ON METER IN FT.
LOSS OF HEAD IN FT.
0
1.00
2.00
3.00
4.00
5.00
6.00
7.00
.10
.20
.30
.40
.50
.60
.70
.80
.90
1.00

FIG. 13.

greatly different from the values for large riveted pipe given in Table No. 4.

In regard to the applicability of Kutter's formula it is to be said that the experiments on the wooden pipe herein described give values of n ranging from 0.012 to 0.015, an average value being perhaps 0.013. The difference between this value and those given for the Denver and Butte City conduit can hardly be attributed to the greater roughness of the Ogden pipe. It is rather to be supposed that the Kutter formula is defective.

IV.—Observations on Venturi Meters.

The Venturi meters being the only means available for measuring the rate of discharge of the pipe, no test could be made of the correctness of their indications, or for the determination of values of the coefficient to be used with them. As already stated, the authors assumed the correctness of the coefficients furnished by the manufacturers.

A matter of some importance, both theoretically and practically, is the loss of head caused by friction, and by the contraction and expansion of the stream within the meter. The values of this loss for different values of the rate of discharge were fairly well determined within the range of the experiments.

Loss of Head in Meters.

The results of the observations of loss of head in the meters have been tabulated for the two meters separately, Table No. 8 referring to the south meter, and Table No. 9 to the north meter. In each case the observations have been arranged in the order of increasing values of the rate of discharge. These tables show values of "head on Venturi," loss of head in meter, and rate of discharge of meter, for each observation. The values in column 2 are found by taking the difference between the readings of the mercury columns communicating with the throat and up-stream sections of the meter; while the values in column 4 are found in the same way from the up-stream and down-stream sections. The values in columns 3 and 5 are found from those in 2 and 4, respectively, by applying the factor 12.6, or $e - 1$, e being the specific gravity of mercury. The values in columns 2 and 6 are repeated from Tables Nos. 1 and 5.

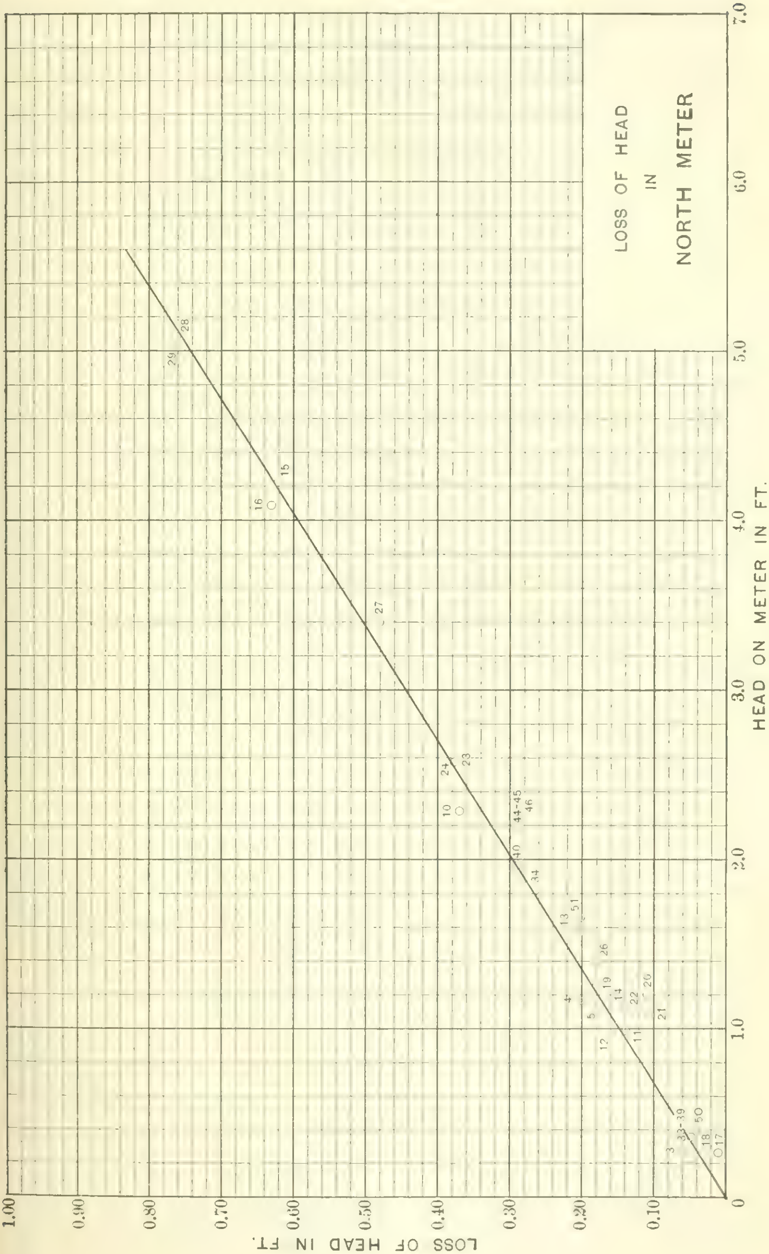

LOSS OF HEAD
IN
NORTH METER
HEAD ON METER IN FT.
LOSS OF HEAD IN FT.
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00

Fig. 14.

TABLE No. 8.—SOUTH VENTURI METER. RELATION BETWEEN RATE OF DISCHARGE AND LOSS OF HEAD IN METER.

1	2	3	4	5	6
Observation No.	Head on Venturi.		Loss of Head in Meter.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
1	.022	.28	.001	.01	14.5
35	.024	.30	.005	.06	15.3
47	.025	.32	.005	.06	15.6
25	.026	.33	.004	.05	16.0
41	.036	.46	.007	.09	19.1
20	.043	.54	.006	.08	21.0
30	.054	.68	.008	.10	23.4
26	.072	.91	.012	.15	27.2
48	.120	1.51	.021	.27	34.9
36	.122	1.54	.021	.27	35.2
27	.123	1.55	.018	.23	35.3
37	.125	1.58	.019	.24	35.6
12	.140	1.76	.016	.20	37.5
11	.141	1.78	.017	.22	37.6
42	.145	1.83	.021	.27	38.2
21	.159	2.00	.023	.29	40.0
22	.160	2.02	.024	.30	40.1
31	.178	2.24	.026	.33	42.5
2	.199	2.51	.026	.33	44.8
29	.210	2.65	.031	.39	46.1
28	.211	2.66	.030	.38	46.2
16	.219	2.76	.033	.41	46.9
15	.221	2.79	.032	.40	47.1
6	.225	2.84	.031	.39	47.5
7	.225	2.84	.031	.39	47.5
14	.226	2.85	.032	.40	47.5
49	.288	3.63	.045	.57	53.8
51	.292	3.68	.044	.56	54.1
13	.296	3.73	.042	.53	54.5
50	.297	3.74	.046	.58	54.6
43	.321	4.04	.048	.61	56.7
34	.363	4.57	.055	.69	60.1
33	.366	4.61	.055	.69	60.4
32	.376	4.74	.058	.73	61.3
46	.380	4.79	.059	.74	61.6
45	.383	4.83	.059	.74	61.9
44	.390	4.91	.060	.76	62.4
10	.391	4.93	.052	.66	62.5
40	.395	4.98	.059	.74	62.7
39	.400	5.04	.058	.73	63.1
38	.406	5.12	.058	.73	63.6
9	.481	6.06	.065	.82	69.2
8	.512	6.45	.072	.91	71.2

The simultaneous values of "head on Venturi" and "loss of head in meter" are plotted, for each meter separately, in Figs. 13 and 14. The irregularities shown in the diagrams are doubtless due chiefly to the presence of dirt at the top of the down-stream mercury column, as already explained. In spite of these irregularities the results show quite satisfactorily the relation between loss of head and head on Venturi. This relation, within the range of the observations, is well rep-

TABLE No. 9.—North Venturi Meter. Relation Between Rate of Discharge and Loss of Head in Meter.

1	2	3	4	5	6
Observation No.	Head on Venturi.		Loss of Head in Meter.		Rate of discharge. (Cubic feet per second.)
	Gauge reading.	Equivalent water column. (Feet.)	Gauge reading.	Equivalent water column. (Feet.)	
17	.021	.26	.001	.01	14.1
18	.021	.26	.002	.03	14.1
3	.022	.28	.005	.06	14.5
33	.028	.35	.004	.05	16.7
39	.028	.35	.004	.05	16.7
50	.032	.40	.003	.04	18.1
12	.077	.97	.013	.16	28.1
11	.078	.98	.011	.14	28.2
21	.086	1.08	.009	.11	29.6
22	.087	1.10	.010	.13	29.8
14	.090	1.13	.011	.14	30.3
5	.090	1.13	.014	.18	30.3
4	.093	1.17	.016	.20	30.9
20	.095	1.20	.009	.11	31.2
19	.099	1.25	.014	.18	31.7
26	.111	1.40	.014	.18	33.7
13	.127	1.60	.017	.21	35.8
51	.130	1.64	.016	.20	36.2
34	.150	1.89	.022	.28	38.9
40	.156	1.97	.023	.29	39.6
44	.174	2.19	.023	.29	42.1
45	.174	2.19	.023	.29	42.1
46	.174	2.19	.023	.29	42.1
10	.181	2.28	.029	.37	42.8
23	.204	2.57	.030	.38	45.5
24	.205	2.58	.030	.38	45.7
27	.270	3.40	.038	.48	51.8
16	.323	4.07	.050	.63	57.0
15	.333	4.21	.049	.62	57.8
29	.400	5.04	.060	.76	63.1
28	.401	5.06	.060	.76	63.2

resented by a straight line. In other words, the loss of head in the meter appears to be directly proportional to head on Venturi. The loss of head thus appears to vary nearly as the square of the velocity of flow through the meter.

If H denotes head on Venturi and H'' loss of head in meter, the above relation is expressed by the equation $H'' = \alpha H$, where α is a constant. The value $\alpha = 0.149$ for each meter agrees well with the experimental data.

The relation between loss of head in meter and rate of discharge, for the Ogden meters, using the relation $H'' = 0.149H$, is shown by the curve on Fig. 15.

The experience of the authors in these tests has convinced them of the great value of the Venturi meter in experimental work, requiring

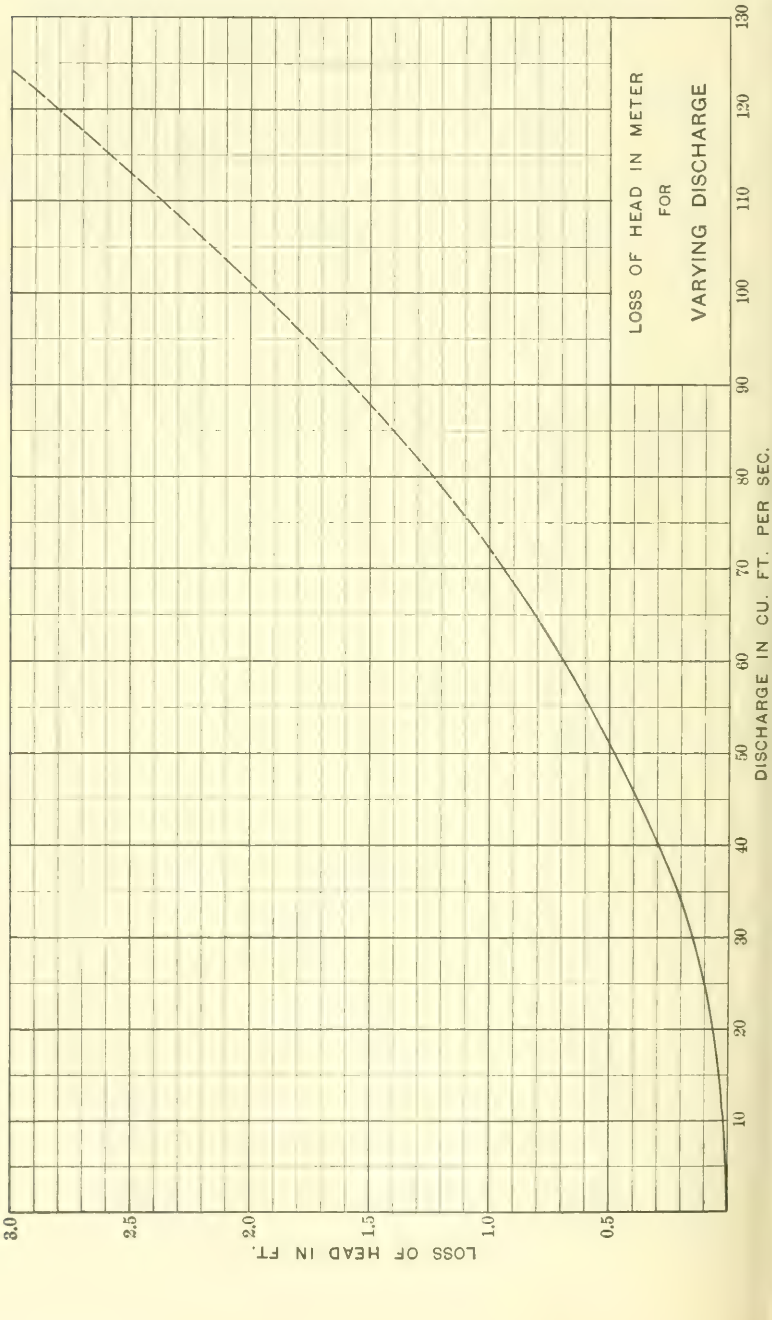

LOSS OF HEAD IN METER
FOR
VARYING DISCHARGE
DISCHARGE IN CU. FT. PER SEC.
10 20 30 40 50 60 70 80 90 100 110 120 130
LOSS OF HEAD IN FT.
0.5 1.0 1.5 2.0 2.5 3.0

Fig. 15.

the measurement of the rate of discharge of pipes. By no other method would it have been possible to collect so much data, at all comparable in accuracy with that obtained in these experiments without months of labor. Moreover, the sensitiveness of the difference-gauge to slight changes of the rate of discharge shows that the meter is capable of giving measurements of great precision. It is perhaps not too much to say, that, except for very small quantities, the Venturi meter furnishes the most precise of all methods of measuring the rate of discharge, provided the values of the coefficient are accurately known.*

In carrying out these experiments the authors were assisted by Mr. L. S. Boggs, E. E., superintendent of the Pioneer Electric Power Company; by Mr. F. W. Hart, assistant superintendent, and by Mr. L. B. Spencer, student in civil engineering in Stanford University. To them, for the valuable and efficient aid rendered; to the directors of the Pioneer Electric Power Company, and especially to Chief Engineer Bannister, for all courtesies rendered, the authors desire to express their sincere thanks.

* In regard to the range of the coefficients of a Venturi meter, the only experimenta results known to the authors are those of Herschel. These results, obtained with meters of diameter 12 ins., 48 ins. and 108 ins., indicate that the values of the coefficient, for throat velocities between 5 ft. and 25 ft. per second, are not likely to differ from unity by more than 3 per cent. (see "115 Experiments," Pl. III, p. 43). Even with an unrated meter it is probable that the results obtained will compare favorably in accuracy with those obtainable by any other method of measurement.

APPENDIX.

Attachment of Piezometers.

Object of discussion.

It is proposed to consider the question whether the height of a piezometer column depends upon the position of the point of attachment to the pipe ; and also whether different results will be given by multiple attachment than by attachment at a single point. The discussion will refer to a vertical cross-section of a horizontal pipe ; but the reasoning will be seen to apply to the transverse section of a pipe having any direction.

Variation of Pressure in a Cross-Section.

The pressures at different points in the same cross-section are unequal, being less near the top of the pipe than near the bottom. If the particles of water all moved in straight lines parallel to the axis of the pipe, the law of variation of pressure with depth would be the same as for water at rest. Any variation from this law must be due to the fact that particles are deflected from the straight axial direction ; in other words, to the fact that some of the particles of water passing a given section at any instant are being accelerated vertically. If such is the case, the distribution of the vertical components of acceleration throughout the cross-section will continually vary ; so that not only will the law of variation of pressure with depth differ from the hydrostatic law, but the actual pressure at any given point will vary. Therefore, in any case in which a piezometer is found to give results which fluctuate but slightly, it may be concluded that the sinuosities in the motions of the particles of water have no important effect upon the law of variation of pressure in the cross-section. Experience bears out the supposition that this condition is satisfied when steady flow has been maintained for some time in a straight pipe of uniform cross-section. The following discussion will therefore proceed on the assumption that the pressure in any cross-section varies with the depth according to the hydrostatic law.

Case of Open-Topped Piezometer.

If the pressure throughout a cross-section exceeds that of the atmosphere, and if open tubes are attached at two points, as at A and B (Fig. 16), it is evident that water will rise to the same level in both. For since the pressure at B exceeds that at A by an amount equivalent to the height (h) of A above B, the corresponding pressure columns must differ in height by the same amount h. In fact, the pressure at any two points C and D at the same level in the two pipes must be equal.

If the pressure at the top of either column is less than atmospheric, that column will stand correspondingly higher ; but the sur-

faces X and Y will be at the same level so long as they are subject to equal pressures.

Case of Vacuum Gauge.

In case the pressure throughout the cross-section of the pipe becomes less than one atmosphere, the open piezometer becomes inapplicable. It is, however, instructive to consider what will happen if, with open tubes attached as in Fig. 16, the pressure within the

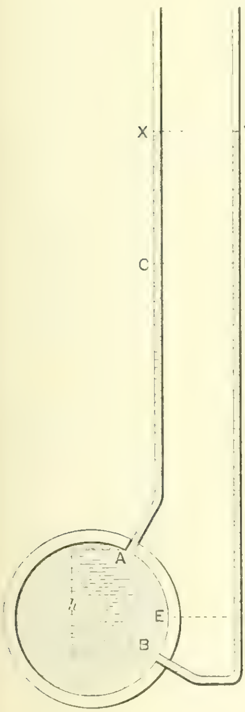


Fig. 16.

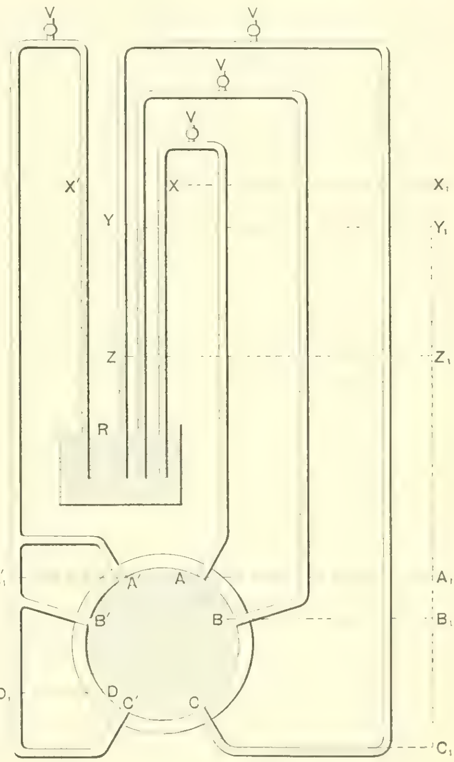


Fig. 17.

pipe gradually diminishes. The surfaces X and Y will fall equally until the pressure within the pipe at the level A becomes just one atmosphere, the surface Y then standing on a level with A. The pressure at any point lower than A is still greater than one atmosphere. Let the pressure in the pipe diminish still further until there is just one atmosphere of pressure at some level E, between A and B; pressures less than one atmosphere existing at all points higher than E, and greater than one atmosphere at all points lower. The surface Y

will fall to F, level with E; while at A air will be drawn into the pipe. If the level of atmospheric pressure falls below B, air will also enter at B.

Suppose next that the tubes attached at A and B are arranged as shown in Fig. 17, both tubes dipping into a reservoir whose surface is under atmospheric pressure, the pressure within the pipe being less than atmospheric in all parts of the cross-section. At first let air be admitted to the tubes through the valves V V, which are then closed. So long as air enters freely, the tubes act in the same way as the open tubes of Fig. 16. The pressure within the piezometer tubes being greater than that within the pipe, air enters the pipe from both tubes. After the valves V V are closed, the withdrawal of air causes a decrease of pressure in each tube, and water rises from the reservoir to a height which, for each tube, represents the difference between the pressure within the tube and that of the atmosphere at the surface of the reservoir. Air will continue to enter the pipe at A so long as the pressure within the tube exceeds the water pressure at A. A condition of equilibrium will finally be reached in which the pressure throughout the air in the tube is equal to the water pressure at A, and the surface X stands at a height above the reservoir surface representing the excess of atmospheric pressure above the pressure within the space A X. A like condition will be reached in the other tube, but since the pressure-head at B exceeds that at A by an amount A_1 $B_1 = h$, the surface Y will stand lower than X by an amount $X_1 Y_1 = h$.

It is further to be noticed that, if the opening at A is of appreciable size, water will stand in the tube up to the level of the top of the opening. Thus, suppose the exhaustion of the air within the tube has proceeded until the air pressure in the tube is greater than the water pressure at the top of the opening and less than that at the bottom; air will enter at the top, and water will flow out at the bottom, a condition of equilibrium being reached only when no part of the stream in the main pipe flowing past the orifice is in contact with air. Similarly, water will stand in the other tube up to the level of the top of the opening B.

It thus appears that it makes no difference at what point of the cross-section the piezometer is attached, provided the indicated pressure be referred to the proper level. If the apparatus is attached and operated in the manner above described, the piezometer reading in each case gives the pressure within the pipe at points which are on a level with the top of the orifice leading from the pipe to the piezometer tube.

If, after the condition of equilibrium above described has been reached, the pressure within the pipe increases, but still remains less than that of the atmosphere, water will rise in the tubes at A and at B, the surfaces X and Y at the same time falling. A condition of

equilibrium will ensue in which the height of the column RX exceeds that of the column above A by an amount equivalent to the difference between atmospheric pressure and the pressure at A, with a similar condition of affairs in the tube attached at B. In such a case both columns in the same piezometer tube must be observed in order that the pressure at a given level in the main pipe may be known.

A further case to be considered is that in which some part of the piezometer tube is lower than the opening into the pipe, as at C, Fig. 17, the pressure within the pipe at C being supposed less than one atmosphere. If the tube is at first filled with air under atmospheric pressure, air will enter the pipe at C until the pressure within the tube falls to the value of the water pressure at the bottom of the opening. As soon as the air pressure becomes less than this value, water will flow into the tube through the lower part of the orifice, collecting in the lowest part of the tube until its cross-section is entirely filled. Water will continue to flow into the tube at the bottom of the orifice, and air to enter the pipe at the top, until no air remains in contact with the water flowing past the orifice in the main pipe. A condition of equilibrium will finally be reached in which water fills the tube from C to some point C_1. This point C_1 will be high enough so that the lowest cross-section of the tube is completely filled, but its exact height cannot be predicted. The height of the column $R\,Z'$ will measure the amount by which atmospheric pressure exceeds the pressure within the pipe at the level C_1.

Consider next the case of a single piezometer communicating with the given cross-section of the pipe at two points, as at A' and C', Fig. 17, the pressure at every point of the cross-section being less than that of the atmosphere. As before, let the valve V be first open and then closed. While it is open, air will enter the pipe at A' and at C'. After it is closed air will continue to enter the pipe so long as the pressure within the tube exceeds the water pressure, either at A' or at C'. At the same time water will rise from the reservoir R to a height measuring the amount by which the pressure within the tube is less than one atmosphere. When the pressure within the tube becomes less than that in the pipe in the lower portion of the opening C', air will cease to enter at this part of the opening, and water will flow into the tube. At the top of the opening air will still enter until the air pressure falls below the water pressure at that point; after which water will flow out through the entire opening. Air will continue to enter the pipe at A', and the air pressure in the tube to decrease. When the pressure in the tube is equal to the water pressure at some point $B\,D$, between A' and C', water tends to stand in the tube at the height D_1; no condition of equilibrium is reached, however, so long as air continues to enter at A'. Finally the pressure in the tube will reach the value of the water pressure at the highest point of the orifice A', air will cease to enter

the main pipe, and water will stand in both branches of the tube to the level $A' \, A'_1$. The air pressure in the tube is now equal to the pressure in the water at the level A', and is less than atmospheric pressure by an amount equivalent to the water column $R \, X'$.

Summary of Results.

The conclusions to be drawn from the foregoing discussion may be summarized as follows:

(1) When the pressure in the given cross-section of the pipe everywhere exceeds that of the atmosphere, an open piezometer will stand at the same height at whatever point of the cross-section it be attached, and whether it communicates with the pipe at one point or at several.

(2) If a "vacuum" piezometer be used, the pressure at the point of attachment being less than that of the atmosphere, the pressure at any level in the pipe can be inferred if the water surfaces in both branches of the piezometer are known. The pressure within the main pipe, at all points on a level with the surface of the column in the branch of the piezometer tube adjacent to the pipe, has "negative" value measured by the height of the column in the other branch above the open reservoir.

(3) In case of a vacuum piezometer connected with the pipe at more than one point in the same cross-section, water will stand at the same level in all the connecting tubes.

(4) In case of a vacuum piezometer, if air be freely admitted to the tube before the observation, the pressure within the main pipe remaining constant, water will rise in the connecting tubes, whether one or several, to the level of the highest point of communication with the pipe.*

The authors have not been able to subject these conclusions to the test of experiment. They are, however, in conformity with the results noticed and discussed by Herschel.† For measuring pressures in the throat of a Venturi meter he employed a vacuum piezometer communicating with the throat at several points, the several branch tubes being furnished with valves, so that communication could be opened through any one or more of them at pleasure. From the working of the piezometer with different combinations of tubes open, Herschel concludes that:

> "The Venturi must, in all cases, be pierced for connection with the air chamber vertically at its crown, and may be pierced radially at as many additional points as we please, without affecting the reading of the standard crown orifice."

The foregoing theory suggests no reason for the conclusion that the pipe must be pierced at the highest point of the cross-section; but

* Unless the tube leading to this point runs below the level of the opening into the pipe.

† *Trans. Am. Soc. C. E.*, Vol. xvii, pp. 248, 250, 251.

Herschel's conclusion is otherwise in strict conformity with the principles reached above, and summarized in the paragraphs numbered (3) and (4). These principles are not fully tested by the experiments recorded, since in every case bearing upon the point under discussion one of the open passages was the one at the top of the cross-section.

For the case of pressures greater than atmospheric, Herschel's experiments of October, 1887,* furnish several instances in conformity with the conclusion above drawn and stated in paragraph (1). This is, in fact, the conclusion reached by Herschel. In the experiments of the authors at Ogden no pressures less than atmospheric had to be measured.

The foregoing discussion has not referred to the effect of air carried by the water and collecting in the piezometer tube. Attachment of the piezometer at the top of the pipe probably gives greater liability to trouble from this source than attachment at a lower point; but in all cases means must be provided for getting rid of whatever air may collect.

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* *Trans. Am. Soc. C. E.*, Vol. xvii, p. 250.

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PAPERS.

THE DETERMINATION OF THE SAFE WORKING STRESS FOR RAILWAY BRIDGES OF WROUGHT IRON AND STEEL.

By E. HERBERT STONE, M. Am. Soc. C. E.

TO BE PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

INTRODUCTORY.

From the date of the first introduction of railways, it was for many years the custom, in the design of a railway bridge, to treat the Moving Load of the train in precisely the same manner as the Fixed Load of the structure itself. The load due to the weight of the train was simply added to the load due to the weight of the structure (with flooring, ballast, permanent-way, etc.), and the section of iron to be used calculated for a working stress of about one-fourth of the assumed breaking stress. This, for wrought iron in tension, allowed a nominal unit stress of 5 tons per square inch.

Later on, text-books, dealing theoretically with the subject, taught that the effect produced by Live Load was just double that produced by Dead Load, and recommended that the two kinds of load should be treated separately; a factor of safety of 6 being used with the former and 3 with the latter. Taking the breaking stress of wrought iron

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

in tension as 21 tons per square inch,* the safe working stress per square inch was given as—

Dead Load.......................................7 tons.
Live Load.......................................3½ "

It was not recognized that the theoretical "Live Load," on which these considerations were based, was one which was never dealt with in practice, and was, moreover, essentially different in its action and mode of application from that due to a locomotive at speed. Still this system showed a great advance on what had gone before, and had the effect of directing attention to the matter. The result was that, although engineers generally regarded this as a theoretical rule to which it would not be prudent to work fully in practice, it became common to limit the stress on tension bars to 4 tons per square inch for live load, and it was generally acknowledged that for dead load the stress per square inch in tension might safely be taken at 6 tons. The Board of Trade limit of 5 tons, however, precluded any improvement in the latter direction.

This may be regarded as the first recognition of the principle that Moving Load may be equated with Fixed Load by the use of a suitable coefficient; but at this stage the coefficient was supposed to be always the same, irrespective of the relative amounts of the two kinds of load to be dealt with.

As wrought-iron bridges of large span became more common, engineers grew to be familiar with the fact that, with the comparatively great mass of metal in a large-span bridge, the effect of a train is relatively less than with a small-span bridge. In view of this fact it became usual in the practice of some of the leading bridge engineers to adopt a scale by which the unit stress allowed was made to vary with the span. It was thus acknowledged that the effect of Moving Load as compared with Fixed Load was not always the same, but became relatively less as the proportion of Fixed Load to Moving Load became greater.

The principle of graduating or apportioning the unit stress to the relative amount of Moving Load work to be done soon became gener-

*For the purposes of this paper it is assumed that the static breaking stress per square inch, for material as ordinarily used for bridge girders, may be taken as 21 tons for wrought iron and 27 tons for steel. Also, that the safe working stress may be taken as one-third of the breaking stress. Hence, for an entirely static or fixed load, the figures adopted are as follows:

	Iron.	Steel.
Breaking stress in tons per square inch....................	21.00	27.00
Safe working stress in tons per square inch...............	7.00	9.00

ally recognized; and in America the application of this principle to practical purposes received the most careful study by many of the bridge engineers of that country. Under the special allotment system, as developed by American engineers, the members of a bridge truss are grouped or classified according to the relative amount of Moving Load work they have to bear, depending on the span of the bridge and the position of the member in the truss. To each group is then assigned a percentage by which the nominal stress (treated as static) is to be increased to obtain the equivalent effective working stress; the permissible stress per unit of area remaining constant. In more recent specifications similar results are obtained by using the nominal stress (treated as static), and allotting to each group a different permissible stress per unit of area.

Meantime the results of the magnificent series of experiments by Wöhler and Bauschinger had been ably dealt with by Launhardt, Weyrauch, Gerber and others, and bridge engineers began to consider the cumulative effect of frequent repetitions of the Moving Load.

Thus, while forty years ago a ton of Moving Load received no more consideration than a ton of Fixed Load, its disposal now is recognized as involving very complex considerations. It is found that for any member of a bridge the immediate effect of the application of a ton of Moving Load is always greater than that produced by a ton of Fixed Load, and that the degree or percentage by which it is greater varies for different ratios of Moving Load to Fixed Load. Similarly, that for repeated loading and unloading, the ultimate cumulative effect, on any member of a bridge, of a ton of Moving Load, is always greater than the immediate effect, and that the degree or percentage by which it is greater varies for different ratios of the initial stress or Fixed Load.

The Nature of the Effects.

Definitions.—The terms "Dead Load" and "Live Load" have already been appropriated in a certain definite sense by the text-books; and in this sense have a special relative value, the effect of Live Load being exactly double that due to Dead Load. For the sake of distinction therefore in this paper the terms "Fixed Load" and "Moving Load" will be used with meanings respectively as follows:

Fixed Load.—By this term is to be understood a load which may for practical purposes be considered absolutely constant, subject to no

movement, vibration or variation. The self-supported weight of the structure itself is thus throughout this paper treated as "Fixed Load" and also any other load supported by the structure which is stationary or which generally remains in position and is not frequently applied or removed. Thus, for example, the weight of the girders of a bridge would be treated as Fixed Load, so would also the ballast, sleepers, permanent-way, etc.*

Moving Load.—By this term is to be understood a load which is subject to variation or which is frequently applied and removed. The effect of the violence or vibration with which the application of the load is accompanied is considered as an enhancement of the effect due to the mere weight of the load. Thus a dead engine, pulled over a bridge by a rope from a crab winch beyond, would be treated as Moving Load; the same locomotive traversing the bridge at full speed would be considered as having an enhanced effect due to the violence and vibration with which its application was accompanied.

The "Live Load" of the Text-Books.—In the case of a girder, the "Live Load" of the text-books would mean a load suddenly applied in a vertical direction, but without impact. This might be represented by a load just touching the upper surface of the girder, but having its weight entirely supported by a cord. If that cord were instantaneously severed, the load would then act as "Live Load" in the ordinary text-book meaning of the term. The same load, unsupported by the cord, and having its whole weight resting on the girder without vibration, would represent "Dead Load."

It can easily be shown that the effect on the girder of the "Live Load" of the text-books is theoretically double the effect of the corresponding "Dead Load."

Live Load Contrasted with Train Load.—Those writers who apply the theoretical "Live Load" of the text-books to the solution of this problem, assume that the effect produced by the train traversing the bridge in a horizontal direction may be taken as the dynamic equivalent of a load suddenly applied in a vertical direction; and that the faster the engine moves, the more nearly does it become "Live Load," in the text-book meaning of the term.

It will, however, be evident on consideration that these two modes of applying the stress are in every respect essentially different. In

* For the purpose of experiment on the relative effect of Fixed Load and Moving Load, an engine standing on a bridge is considered as Fixed Load, the same engine traversing the bridge at full speed being taken as Moving Load.

the case of the "Live Load" of the text-books a load is applied vertically by the steady action of gravity, while in the case of the Moving Load on a railway bridge there is a locomotive engine running horizontally by means of its own self-developed power, with all the plunging and peculiar vibration contingent on the special construction of the machine, the irregularities of the permanent-way, and the deflection of the girders.

An engine standing at rest on a bridge is certainly "Dead Load" in the ordinary text-book meaning of the term; but when that engine begins to move, the effect on the bridge, due to gravity, is reduced instead of being increased. If the effects of vibration and deflection be neglected, and the engine be supposed to run with absolute smoothness, it is clear that the faster the engine moves in a horizontal direction, the less will be its effect in a vertical direction, and that if the speed became infinite the effect due to gravity would become nothing.

The only way in which a train passing on to a bridge in a horizontal direction could be made approximately to represent a load suddenly applied in a vertical direction would be to cause the train to run at a great speed on to the bridge and stop suddenly in the middle.

It is further to be noted that whereas the "Live Load" of the text-book rule means a load suddenly applied, but without shock or violence, the Moving Load of a locomotive differs essentially therefrom; its effect being less on one hand, in that it is not suddenly applied, while it is greater on the other hand, in that its application is always accompanied by a certain amount of vibration, which is in some cases of great violence.

It is evident, therefore, that a consideration of the "Live Load" of the text-books will not help in any way toward a knowledge of the effect produced on a bridge by the Moving Load of a locomotive. The two kinds of load are essentially different in their nature and in their mode of application; the effect produced by the locomotive may be greater than that of the theoretical "Live Load," or it may be less, and should it in any case be found by experiment to be the same, this can only be regarded as a coincidence.

Two-Fold Effect of Moving Load.—When a locomotive at full speed traverses a bridge, there is an immediate measurable effect produced,

which is greater than that due to the same load at rest. This immediate extra effect is due to the sudden and violent manner in which the load is applied, the result being an increased deflection of the structure as a whole, accompanied by a general jarring and vibration, which is especially noticeable in the smaller and lighter members.

It has, moreover, been established, by the experiments of Wöhler, Bauschinger and others, that the immediate effect of the Moving Load is by no means the whole effect, but that frequent repetitions of stress have a cumulative effect on the structure, of very great importance; and it has been ascertained that a bar subjected to stress, repeatedly applied and removed, will ultimately break with a load very much less than it would have borne for a few applications only.

Thus there are two distinct effects of the Moving Load to take into account, and it appears necessary to draw special attention to this point, as it is one which is often lost sight of, even by good authorities.

On one hand it is sometimes found that the "Cumulative Effect" is ignored, and it is argued that on a large-span bridge the effect of the Moving Load is practically no greater than that of the same weight as Fixed Load, because the observed extra deflection is but trifling. On the other hand it is frequently found that the "Immediate Effect" is ignored, and the deductions from Wöhler's experiments by Launhardt, Weyrauch and others are discussed as if cumulative effect were the only matter for consideration.

To determine a coefficient for the Moving Load by a study of cumulative effect alone would be to assume the effect on a bridge of an engine traveling at sixty miles an hour to be no more severe than that due to the same engine "dead" hauled over slowly with a rope from a winch beyond the abutment. On the other hand to determine a coefficient for the Moving Load by a study of the immediate effect alone, would be to assume the effect on a bridge of an engine traversing the structure an indefinite number of times to be no greater than that produced by the engine traversing the structure only once.

In this paper, therefore, the effect of the Moving Load will be considered under two heads, viz.: (*a*) Immediate Effect—which is observable every time a train passes over the bridge; and (*b*) Cumulative Effect—produced in course of time by repeated loading and unloading.

Immediate Effect of the Moving Load.

The Measure of Immediate Effect.—If a locomotive be run slowly on to a bridge and there brought to a stand, the stresses produced will be simply those due to the weight of the machine as Fixed Load. If the engine be now made to traverse the bridge at a considerable speed, the immediate effect of the extra stress produced is indicated by an increased deflection of the structure as a whole, and by an increased elongation or shortening of members in tension or compression.

These effects may be measured by suitable instruments, and the immediate effect produced by the locomotive in motion can be compared with that produced by the mere weight of the machine at rest. Thus, for example, if the deflection of a girder with the engine at speed is found to be half as much again as that due to the dead weight of the engine at rest, evidently this same extra deflection could have been produced with the engine standing on the bridge by loading it with pig iron until the weight resting on its wheels had been increased by 50 per cent.* A ton of Moving Load in this case being found to produce on the bridge an immediate measurable effect as severe as one and a half tons of Fixed Load, it is clear that for the purpose of determining the stresses the actual weight of the Moving Load would have to be multiplied by a coefficient of 1.5, to obtain its equivalent in terms of Fixed Load.

Observations in India on Deflection.—Since the year 1879 deflection observations have been regularly made in India by the Government Inspector on the bridges of each new section of railway before it is opened for traffic. These observations are, whenever practicable, made with self-recording apparatus, and are conducted on a uniform system under Government rules. The author has made an analysis of all the results thus obtained for ten years, from June, 1882, to June, 1892. This deals with nearly 1 500 separate observations made by a number of different inspecting officers in all parts of India where railways had been constructed during that period, and with every variety of girder in ordinary use, over about 7 500 miles of new railway. Some of these lines are on the 5 ft.-6 in. gauge, others on the meter-gauge. Some are owned by the Government, others by companies.

It will be seen, therefore, that the results thus obtained represent averages, not only of a considerable number of actual observations,

* This, of course, depends on the assumption that the measurable deflection increases directly as the actual stress.

but also for a variety of different spans and different designs. Further, that the experiments were conducted on a uniform system and under practical conditions by a number of entirely independent observers. No doubt, in many cases the original data may have been but roughly determined, and the observers perhaps sometimes did their work in a careless or perfunctory manner; but any errors due to defective observation are as likely to be on one side as the other, and with the large number of observations on which the averages were taken, and the large number of different observers who contributed to these averages, it is probable that the results obtained fairly represent the tendency of the actual effects produced under ordinary conditions.

Collation of the Results.—In collating and arranging these data the average of all the observations was first taken for each span separately, and the results so obtained plotted by points on the diagram. A note was then made at each point to show the number of observations on which the position of that point depended, and each point in succession was joined to that immediately beyond it by a straight line. On each straight line so obtained a mark was made dividing it into two parts in the inverse ratio of the numbers at its end points. These marks thus again averaged the averages previously obtained, in the ratio of their relative importance. It will be seen, therefore, that under this system each point was given votes (as it were) in proportion to the number of observations it represented, and the position in which the curve was finally drawn was determined accordingly. Through the last point, thus obtained experimentally for the greatest proportion of Fixed Load, the line was continued as an even curve to pass through the point of all "Fixed Load" where the immediate effect of the Moving Load becomes nothing. The result thus obtained is illustrated in the diagram, Fig. 1.

Endeavors were made, by different systems of plotting and in other ways, to discover some law or rule by which the form of the curve might be determined, but without success. A parabola was, of course, first suggested, but was found quite irreconcilable by any reasonable concessions. The result now presented, therefore, embodies no theory; it is merely an even curve averaging the actual results as nearly as practicable.

Results Accepted for Practical Purposes.—To what extent the results now obtained for girders as complete structures apply to individual

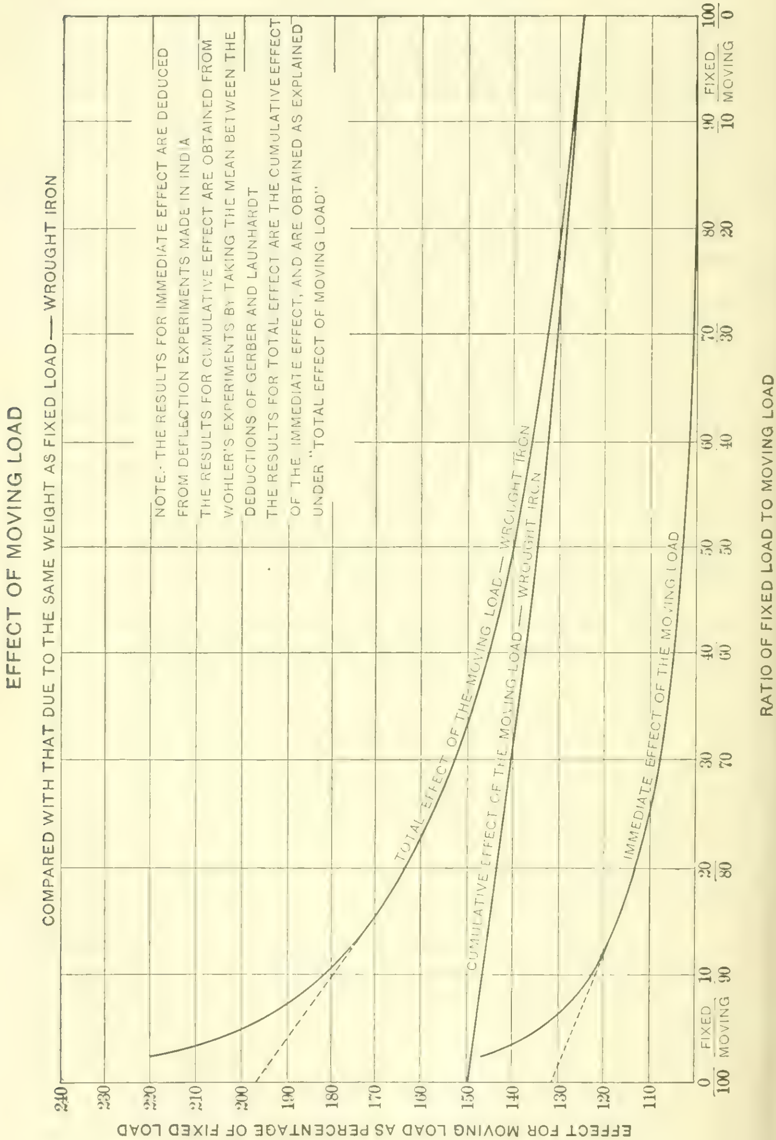
EFFECT OF MOVING LOAD
COMPARED WITH THAT DUE TO THE SAME WEIGHT AS FIXED LOAD — WROUGHT IRON
NOTE.- THE RESULTS FOR IMMEDIATE EFFECT ARE DEDUCED FROM DEFLECTION EXPERIMENTS MADE IN INDIA
THE RESULTS FOR CUMULATIVE EFFECT ARE OBTAINED FROM WOHLER'S EXPERIMENTS BY TAKING THE MEAN BETWEEN THE DEDUCTIONS OF GERBER AND LAUNHARDT
THE RESULTS FOR TOTAL EFFECT ARE THE CUMULATIVE EFFECT OF THE IMMEDIATE EFFECT, AND ARE OBTAINED AS EXPLAINED UNDER "TOTAL EFFECT OF MOVING LOAD"
TOTAL EFFECT OF THE MOVING LOAD — WROUGHT IRON
CUMULATIVE EFFECT OF THE MOVING LOAD — WROUGHT IRON
IMMEDIATE EFFECT OF THE MOVING LOAD
EFFECT FOR MOVING LOAD AS PERCENTAGE OF FIXED LOAD
240
230
220
210
200
190
180
170
160
150
140
130
120
110
RATIO OF FIXED LOAD TO MOVING LOAD
FIXED / MOVING
0/100
10/90
20/80
30/70
40/60
50/50
60/40
70/30
80/20
90/10
100/0

Fig. 1

truss members can only be determined by an extended series of detailed experiments on individual members in girders of different types; but as the law by which the effects are governed must be the same in both cases, it is probable that the scale by which the value of the coefficient varies, for different ratios of Moving Load to Fixed Load, will be of a similar character; and as the results obtained from the deflection of the truss, as a whole, must to a great extent represent the average for all its members, it will be evident that the error cannot in any case be of great importance.

It is probable that a complete series of experiments, carefully conducted with efficient apparatus, would indicate that each class of members of a triangulated girder has its own proper series of coefficients, and that another series would apply to plate girders. If the lines representing each of these series of coefficients were plotted on a diagram, a maximum and minimum boundary line could then be drawn, enclosing a strip or band within which all the coefficient lines would lie.

Remembering that herein only the coefficient for immediate effect is being discussed—that this has to be combined with the cumulative effect to obtain the total effect—that this total effect is for Moving Load alone, and that to obtain the total effective stress on the bar the Fixed Load is to be added—remembering, further, that the safe working stress on a bar, due to all these effects combined, is only one-third of the ultimate breaking stress for that bar—it will, no doubt, be agreed that the width of the strip or band, containing the various coefficient lines, is not likely to be great enough to affect to any important extent the actual dimensions which would be given to any individual member of a truss as a result of these calculations. Still less would it be likely to affect the approximate formula recommended for general use.

For practical purposes, therefore, and in the absence of more complete data, it is considered that the results of the deflection experiments made in India may be accepted as a sufficiently close approximation to the average immediate effect produced on the members of a bridge truss by a passing train.

The general results for the immediate effect of the Moving Load, as obtained from these experiments, are given in Table No. 1*, for different

*For the purposes of this paper it is assumed that the static breaking stress per square inch, for material, as ordinarily used for bridge girders, may be taken as 21 tons for wrought iron and 27 tons for steel. Also, that the safe working stress may be taken as one-third of the breaking stress. Hence, for an entirely static or fixed load, the figures adopted are as follows:

	Iron.	Steel.
Breaking stress in tons per square inch	21.00	27.00
Safe working stress in tons per square inch	7.00	9.00

conditions of loading, from "All Moving Load" advancing by 5% differences up to "All Fixed Load." These results are also exhibited in a graphic form in Fig. 1.

TABLE No. 1.—IMMEDIATE EFFECT OF MOVING LOAD. DEDUCED FROM DEFLECTION EXPERIMENTS MADE IN INDIA.

Nature of Compound Load. Ratio Percentage.		Immediate effect of Moving Load compared with that of Fixed Load. Percentage.	Immediate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	......	.....	....	.....
2.5	97.5	147.40	14.36	0.36	14.00
5	95	133.78	15.90	0.79	15.10
10	90	122.63	17.45	1.74	15.70
15	85	116.86	18.37	2.76	15.61
20	80	113.06	19.01	3.80	15.21
25	75	110.27	19.50	4.87	14.62
30	70	108.13	19.87	5.96	13.91
33.3	66.7	107.14	20.05	6.68	13.37
35	65	106.63	20.13	7.05	13.09
40	60	105.47	20.33	8.13	12.20
45	55	104.62	20.48	9.22	11.26
50	50	103.87	20.60	10.30	10.30
55	45	103.23	20.70	11.38	9.31
60	40	102.63	20.78	12.47	8.31
65	35	102.10	20.85	13.55	7.30
66.7	33.3	101.96	20.86	13.91	6.95
70	30	101.64	20.90	14.63	6.27
75	25	101.21	20.94	15.70	5.23
80	20	100.82	20.97	16.77	4.19
85	15	100.53	20.98	17.84	3.15
90	10	100.30	20.99	18.89	2.10
95	5	100.13	21.00	19.95	1.05
100	0	100.00	21.00	21.00	0.00

CUMULATIVE EFFECT OF THE MOVING LOAD.

The celebrated experiments of Wöhler, followed by further observations in the same direction by Spangenberg, Bauschinger and B. Baker, have furnished engineers with a mass of data regarding the cumulative effect produced on a structure by repeated loading and unloading.

The experiments, it must be remembered, were conducted with apparatus specially arranged to apply and remove the load in a steady and uniform manner, the effect of shocks and the violent jarring and hammering action, which is observed when a locomotive traverses a bridge at speed, being entirely eliminated. The results of these ex-

periments on cumulative effect are, therefore, exactly what is required to supplement the observations on immediate effect.

The general results are familiar to all engineers who have interested themselves in the subject, and are briefly as follows:

Let t = The greatest stress the bar will bear under a static load. Condition — "All Fixed Load."
(*Tragfestigkeit*, or Statical breaking strength.)

u = The greatest stress of which the bar will bear an indefinite number of repetitions with the load applied and removed. Condition — "All Moving Load."
(*Ursprungsfestigkeit*, or Primitive strength.)

$+r$ and $-r$ = The greatest stress of which the bar will bear an indefinite number of alternations. The load producing tension and compression alternately; the stresses being equal, but in opposite directions.

a = The actual stress under which the bar breaks—sometimes called the "Working Strength" of the bar.
(*Arbeitsfestigkeit*, or Ultimate working strength.)

Min. S = The least stress to which the bar is subjected.

Max. S = The greatest stress to which the bar is subjected.

$\phi = \frac{\text{Min. } S}{\text{Max. } S}$. This expression also represents the ratio of initial stress or Fixed Load to the Total Load. For example, if a compound load be made up of one-fourth Fixed Load and three-fourths Moving Load, the value of ϕ will be .25.

First.—With a Moving Load applied and removed an indefinite number of times, and alternating from zero to a certain fixed quantity, a bar will ultimately break with a load considerably less than that which it would have been able to bear as Fixed Load. In other words, u is always considerably less than t. The ratio of $u : t$ is found to vary with different materials, and some of the experiments show great discrepancies; but, in a general way, it appears that the difference between u and t is greater in steel with a comparatively large percentage of carbon and high static breaking stress, and less with mild steel having a lower static breaking stress. With wrought iron, moreover, it is generally less than with mild steel.

The actual relative value of u, as compared with t, may be taken roughly as varying from $u = \frac{2}{3} t$ for wrought iron down to $u = \frac{1}{2} t$ for tool steel.

Second.—With a stress alternating from a positive amount, $+ r$, to an equal negative amount, $- r$, the bar will ultimately break if $r =$ about $\frac{1}{2} u$.

Third.—With a certain initial stress or Fixed Load (Min. S), and a certain Moving Load added and removed (causing the total stress to vary by loading and unloading from Min. S to Max. S), the stress on the bar becomes compound, being partly due to Fixed Load and partly due to Moving Load. Under these conditions, the total stress a under which the bar ultimately breaks will vary with the ratio or percentage of Fixed Load to Moving Load.

The extreme limits are: on one side "All Fixed Load," when

Range, Min. S to Max. $S = 0$, then $a = t$ and $\phi = 1$;

and on the other side, "All Moving Load," when

Range, Min. S to Max. $S = u$, then $a = u$ and $\phi = 0$.

Between these limits the value of a for any ratio of Fixed Load to Moving Load can be determined by means of "Gerber's parabola," or by the use of Launhardt's formula.*

Gerber's Parabola.—The range of stress which a bar will bear for an indefinite number of repetitions of the load applied and removed having been determined by Wöhler's experiments for different conditions of loading, it was found by Gerber that if the ranges of stress be plotted as ordinates, and the corresponding minimum stresses as abscissæ, the points would fall approximately on a parabolic curve.

Let $\triangle =$ The range of stress, *i. e.*, Max. $S \mp$ Min. S.†

$t =$ The greatest stress the bar will bear under a static load.

$k =$ A constant for the material.

Then Gerber's equation may be written thus—

$$(\text{Min. } S + \tfrac{1}{2} \triangle)^2 + k \triangle = t^2$$

The ultimate static breaking stress for wrought iron, as ordinarily used for bridge girders, may be taken as 21 tons per square inch. For iron of this quality, the results of Wöhler's experiments show that the

* For an account of other methods of dealing with the subject, which have been proposed by Schäffer, Müller, Winkler, Cain, Smith, Seefehlner, Ritter, Lippold, and Clericetti, with tabular comparisons of results, see the paper by Weyrauch published in *Proc. Inst. C. E.*—1882 83.—Vol. lxxi, p. 298.

† The upper sign is to be taken where the stresses are of the same kind and the lower, if of different kinds (*i. e.*, ranging between tension and compression).

greatest stress of which the bar will bear an indefinite number of repetitions, with the load applied and removed, will be about 14 tons, *i. e.*, about two-thirds of the ultimate static breaking stress. The corresponding value of k in Gerber's equation will then be 28.

The general results for cumulative effect of the Moving Load, obtained by the use of Gerber's Parabola for wrought iron, are given in Table No. 2, for different conditions of loading from "All Moving Load," when :

Range from Min. S to Max. $S = u$, and $a = u$:

advancing by 5% differences up to "All Fixed Load," when,

Range from Min. S to Max. $S = 0$, and $a = t$.

TABLE No. 2.—Cumulative Effect of Moving Load Deduced from Wöhler's Experiments by Gerber's Parabola.

Nature of Compound Load. Ratio Percentage.		Cumulative effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	150.00	14.00	0.00	14.00
2.5	97.5	149.00	14.21	0.36	13.86
5	95	148.00	14.42	0.72	13.70
10	90	146.00	14.85	1.49	13.37
15	85	144.00	15.28	2.29	12.99
20	80	142 01	15.72	3.14	12.57
25	75	140.05	16.15	4.04	12.11
30	70	138.12	16.58	4.97	11.60
33.3	66.7	136.85	16.86	5.62	11.24
35	65	136.22	17.00	5.95	11.05
40	60	134.37	17.41	6.96	10.45
45	55	132.57	17.81	8.01	9.80
50	50	130.81	18.20	9.10	9.10
55	45	129.12	18.57	10.21	8.36
60	40	127.48	18.92	11.35	7.57
65	35	125.91	19.25	12.52	6.74
66.7	33.3	125.40	19.36	12.91	6.45
70	30	124.40	19.57	13.70	5.87
75	25	122.96	19.86	14.90	4.97
80	20	121.58	20.13	16.11	4.03
85	15	120.26	20.38	17.32	3.06
90	10	119.00	20.61	18.55	2.06
95	5	117.81	20.81	19.77	1.04
100	0	116.67	21.00	21.00	0.00

Launhardt's Formula.—The formula proposed by Launhardt to express the results claimed by Wöhler's experiments is as follows:

$$a = u \left(1 + \frac{t-u}{u} \cdot \frac{\text{Min. } S}{\text{Max. } S}\right)$$

It will be observed that if the results obtained by the use of Launhardt's formula be plotted on the same system as that adopted by Gerber, the curve obtained will also be a parabola.

Taking the data for wrought iron as before (see under Gerber's parabola) there results:

For an entirely static load (*i. e.*, All Fixed Load) $t = 21$;

For a load alternating from zero to maximum (*i. e.*, All Moving Load) $u = 14$.

Let $\frac{\text{Min. } S}{\text{Max. } S}$ be represented by the symbol ϕ; then, for wrought iron, Launhardt's formula may be written thus:

$$a = 14 \left(1 + \frac{\phi}{2} \right)$$

The general results for the cumulative effect of the Moving Load, obtained by the use of Launhardt's formula for wrought iron, are given in Table No. 3 for different conditions of loading from "All Moving Load," when:

Range from Min. S to Max. $S = u$, then $a = u$ and $\phi = 0$, advancing by 5% differences up to "All Fixed Load," when:

Range from Min. S to Max. $S = 0$, then $a = t$ and $\phi = 1$.

Total Effect of the Moving Load.

Definition of Total Effect.—The total extra effect produced by the Moving Load, as compared with that due to the same weight as Fixed Load, may be taken as that which would be produced by an indefinite number of repetitions of the immediate effect. In other words, the total effect which the bridge should be designed to bear with safety is the ultimate cumulative effect of the immediate effect.

Method of Calculation.—Let it be assumed, for example, that a certain member of a bridge girder has to be designed to bear a compound load, of which 10% is due to Fixed Load, and 90% due to Moving Load, giving a ratio of—

$$\frac{10 \text{ fixed}}{90 \text{ moving}}$$

and that the actual amount of the Moving Load is 100 tons.

This 100 tons of Moving Load is, owing to the violence with which it is applied, found by experiment (see Table No. 1) to produce an immediate effect as severe as that which would be produced by 122.63 tons applied quietly.

TABLE No. 3.—CUMULATIVE EFFECT OF MOVING LOAD. DEDUCED FROM WÖHLER'S EXPERIMENTS BY LAUNHARDT'S FORMULA.

Nature of Compound Load. Ratio Percentage.		Cumulative effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons Per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	150.00	14.00	0.00	14.00
2.5	97.5	149.38	14.18	0.35	13.82
5	95	148.78	14.35	0.72	13.63
10	90	147.62	14.70	1.47	13.23
15	85	146.51	15.05	2.26	12.79
20	80	145.45	15.40	3.08	12.32
25	75	144.44	15.75	3.94	11.81
30	70	143.48	16.10	4.83	11.27
33.3	66.7	142.86	16.33	5.44	10.89
35	65	142.55	16·45	5.76	10.69
40	60	141.67	16.80	6.72	10.08
45	55	140.82	17.15	7.72	9.43
50	50	140.00	17.50	8.75	8.75
55	45	139.22	17.85	9.82	8.03
60	40	138.46	18·20	10.92	7.28
65	35	137.74	18.55	12.06	6.49
66.7	33.3	137.50	18.67	12.44	6.22
70	30	137.04	18.90	13.23	5.67
75	25	136.36	19.25	14.44	4.81
80	20	135.71	19.60	15.68	3.92
85	15	135.09	19.95	16.96	2.99
90	10	134.48	20.30	18.27	2.03
95	5	133.90	20.65	19.62	1.03
100	0	133.33	21.00	21.00	0.00

Having found the enhancement of stress due to the violence with which the load is applied, the cumulative effect will evidently be that due to an indefinite number of repetitions of this enhanced stress. For cumulative effect, therefore, the load which is applied to and removed from the member every time the train traverses the bridge must be taken as 122.63 tons (instead of 100 tons). The Fixed Load, however, remains as before.

Hence, for the purpose of determining the cumulative effect, the ratio—

$$\frac{10 \text{ fixed}}{90 \text{ moving}}$$

will be changed, the Moving Load being increased by 22.63%, and the ratio will become—

$$\frac{10 \text{ fixed}}{110.367 \text{ moving}}$$

or as a percentage ratio—

$$\frac{8.308 \text{ fixed}}{91.692 \text{ moving}}$$

The coefficient for ultimate cumulative effect, applicable to this ratio by Gerber's Parabola, is 1.467. This coefficient is to be applied to 122.63 tons (not 100 tons) and—

$$122.63 \times 1.467 = 179.9 \text{ tons.}$$

By Launhardt's formula the results would be:

$$122.63 \times 1.480 = 181.5 \text{ tons.}$$

Mode of Application—Hence with a ratio of—

$$\frac{10 \text{ fixed}}{90 \text{ moving}}$$

a Moving Load of 100 tons applied and removed an indefinite number of times would produce a total effect (the cumulative effect of the immediate effect) equivalent to about 180 tons as fixed load. In other words, with this ratio, to obtain the equivalent of the Moving Load in terms of Fixed Load, the coefficient would be 1.8; and, to determine the dimensions of the member, the Moving Load may be multiplied by 1.8, the result added to the actual Fixed Load, and, for the purposes of calculation, the total so obtained used as "All Fixed Load."

Statement of Results.—The general results for the total effect of the Moving Load (*i. e.*, the ultimate cumulative effect of the immediate effect) for wrought iron are given in Table No. 4 for different combinations of loading, from "All Moving Load," advancing by 5% differences, up to "All Fixed Load." The total effect here given is calculated as explained above, the cumulative effect as used for calculation being taken as the mean between the results obtained by the use of Gerber's Parabola and those obtained by the use of Launhardt's formula. These results are also exhibited in a graphic form in Fig. 1.

Results Applied to Practice.

Results by Experiment.—For wrought iron, the actual effects for a Compound Load, as determined by experiment, for different combinations of Fixed Load and Moving Load, are shown on Table No. 4.

It will be remembered that these figures represent the combined results for immediate effect and for cumulative effect, and that for immediate effect the curve was not plotted by any rule, but was merely

TABLE No. 4.—Total Effect of Moving Load. The Ultimate Cumulative Effect of the Immediate Effect.

Nature of Compound Load. Ratio Percentage.		Total effect of Moving Load compared with that of Fixed Load. Percentage.	Ultimate Breaking Stress for the Compound Load. Tons per Square Inch.		
			For the Total Load.	Apportionment of the Breaking Stress.	
Fixed Load.	Moving Load.			Due to Fixed Load.	Due to Moving Load.
0	100	......	......	......	......
2.5	97.5	220.31	9.66	0.24	9.42
5	95	199.04	10.82	0.54	10.28
10	90	180.68	12.17	1.22	10.95
15	85	170.42	13.14	1.97	11.17
20	80	163.16	13.95	2.79	11.16
25	75	157.44	14.68	3.67	11.01
30	70	152.75	15.34	4.60	10.74
33.3	66.7	150.30	15.73	5.24	10.48
35	65	149.07	15.92	5.57	10.35
40	60	145.93	16.46	6.59	9.88
45	55	143.31	16.96	7.63	9.33
50	50	140.90	17.43	8.72	8.72
55	45	138.70	17.89	9.84	8.05
60	40	136.63	18.32	10.99	7.33
65	35	134.71	18.73	12.17	6.55
66.7	33.3	134.13	18.85	12.57	6.29
70	30	132.94	19.11	13.38	5.73
75	25	131.28	19.48	14.61	4.87
80	20	129.73	19.82	15.86	3.96
85	15	128.36	20.14	17.12	3.02
90	10	127.13	20.45	18.40	2.04
95	5	126.02	20.73	19.69	1.04
100	0	125.00	21.00	21.00	0.00

drawn in evenly, to average, as nearly as practicable, the actual values obtained by experiment. The corresponding curve has, therefore, no equation, and the results can only be utilized by a reference to the table itself.

This process would, however, not be specially troublesome, as, in practice, a corresponding table or diagram would be used to facilitate computations made on any modern system, or based on an ordinary formula, such as that of Launhardt. It would, in either case, merely be necessary to ascertain the percentage ratios of Moving Load to Fixed Load, and take the corresponding figures from the table or diagram.

It is, nevertheless, certainly desirable to have a simple rule or easily remembered formula, and the following are accordingly offered for consideration, as giving results sufficiently near to those obtained by experiment, and as being, at the same time, easy of application. The effect of each of these systems, for wrought iron, is exhibited graph-

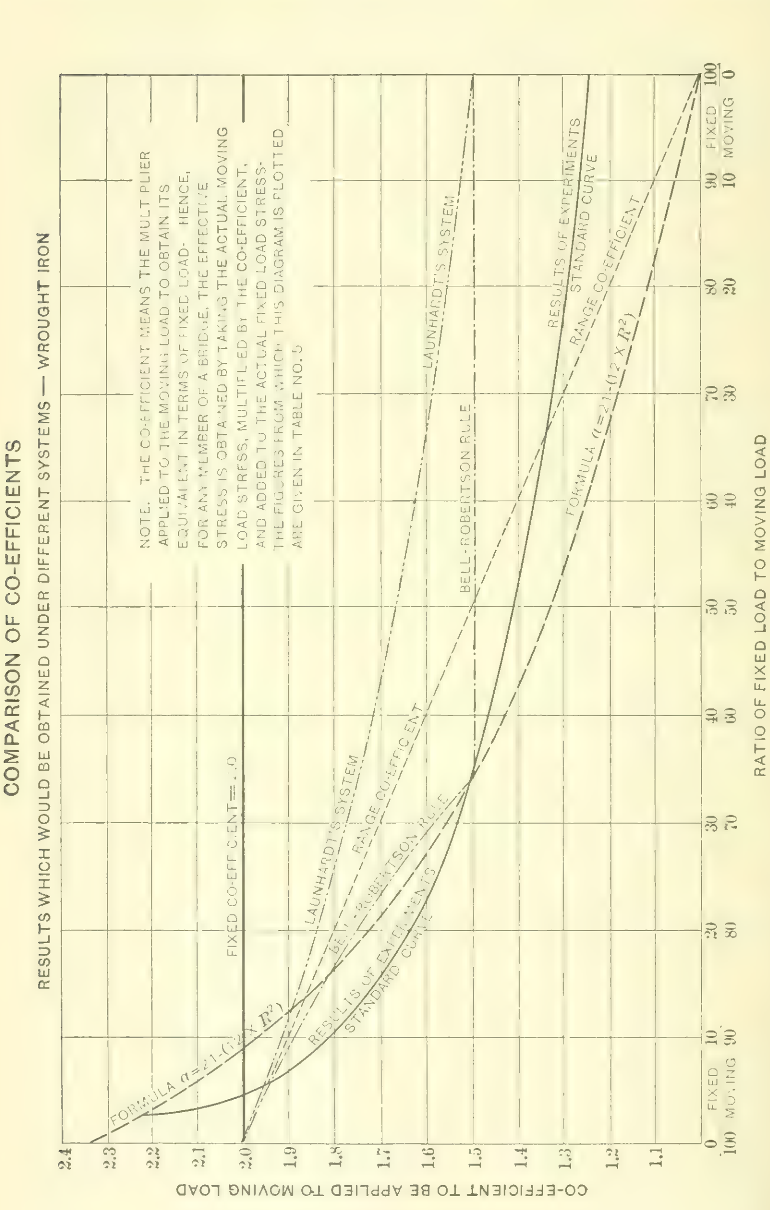
COMPARISON OF CO-EFFICIENTS
RESULTS WHICH WOULD BE OBTAINED UNDER DIFFERENT SYSTEMS — WROUGHT IRON
NOTE. THE CO-EFFICIENT MEANS THE MULTIPLIER APPLIED TO THE MOVING LOAD TO OBTAIN ITS EQUIVALENT IN TERMS OF FIXED LOAD. HENCE, FOR ANY MEMBER OF A BRIDGE, THE EFFECTIVE STRESS IS OBTAINED BY TAKING THE ACTUAL MOVING LOAD STRESS, MULTIPLIED BY THE CO-EFFICIENT, AND ADDED TO THE ACTUAL FIXED LOAD STRESS. THE FIGURES FROM WHICH THIS DIAGRAM IS PLOTTED ARE GIVEN IN TABLE NO. 5
FIXED CO-EFFICIENT = 2.0
FORMULA $a=2.1-(.12 \times R^2)$
RESULTS OF EXPERIMENTS STANDARD CURVE
LAUNHARDT'S SYSTEM
RANGE CO-EFFICIENT
BELL-ROBERTSON RULE
CO-EFFICIENT TO BE APPLIED TO MOVING LOAD
2.4
2.3
2.2
2.1
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
RATIO OF FIXED LOAD TO MOVING LOAD
FIXED 0 10 20 30 40 50 60 70 80 90 100
MOVING 100 90 80 70 60 50 40 30 20 10 0

Fig. 2.

ically in Figs. 2 and 3, for comparison with one another and with the results of experiment. The curve representing the results of experiment is, for distinction, called the "Standard Curve."

General Principles.—It is assumed as established, that, for any Compound Load, the variation in the breaking stress per unit of area depends on the range of stress; that is to say, on the relative amount of Moving Load in the Compound Total Load. To assist in the investigation of this subject, and to enable different systems to be properly compared, it will, therefore, be convenient to adopt a symbol which will directly represent the range. For stresses all in the same direction (*i. e.*, all in tension or all in compression): Range of stress = Moving Load = Max. S — Min. S.

$$\text{Let } R = \frac{\text{Moving Load}}{\text{Moving Load} + \text{Fixed Load}} = \frac{\text{Range}}{\text{Total}}$$

Then the value of R increases directly as the range of stress, and represents the ratio which the range of stress bears to the total stress.

The extreme limits in the value of R are, on one side, "All Fixed Load," when:

$$\text{Min. } S = \text{Max. } S, \text{ then } a = t \text{ and } R = 0,$$

and on the other side, "All Moving Load" when:

$$\text{Min. } S = 0, \text{ then } a = u \text{ and } R = 1.$$

It will be observed that R is thus the complement of the symbol used for the Launhardt formula, thus :—

$$R + \phi = 1, \ R = 1 - \phi, \ \phi = 1 - R$$

and, remembering this relation, it is easy to translate an expression from one system to the other.

Fixed Coefficient.—Under this system each ton of Moving Load is assumed to have a constant relative value, as compared with a ton of Fixed Load, and, on any member of a bridge, the nominal unit stress would merely depend upon how much Moving Load and how much Fixed Load had to be compounded at that relative value.

It will be seen, however, from the results of experiment, that the effect of Moving Load, as compared with that of Fixed Load, is not in all cases the same, but is relatively greater as the proportion of Moving Load becomes higher. Thus, according to the results of experi-

ment, the coefficients proper to different conditions of loading are as follows:

Moving Load.	Fixed Load.	Actual Coefficient.*
10	90	1.27
50	50	1.41
90	10	1.81

It might, therefore, at first sight appear that under the single coefficient system it would not be possible to obtain a graduated scale for the permissible unit stress which should be fairly in accord with the requirements of the case; but it will be found on consideration that under this system the graduation is better than might at first be supposed; and that a single coefficient, if selected as applicable to a high ratio of Moving Load to Fixed Load, would in itself provide a sliding scale yielding results which would perhaps nowhere differ to an important extent from those which would be obtained under a more complete system.

Remembering that the coefficient is applied only to that part of the Total Load which has been called "Moving Load," it is evident that its effect in increasing the size of any member becomes relatively small as the proportion of "Moving Load" to be taken by that member is reduced. Thus, with a single coefficient selected as applicable to a high ratio of Moving Load as compared with Fixed Load, the percentage of error in the Total Load obtained by its use would be decreased roughly in proportion as the actual error in the coefficient itself became greater; and with the maximum error in the coefficient, as in the condition of (practically) all Fixed Load, the error in the Total Load used for the determination of the dimensions of a member would become (practically) nil.

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* As there has been some misunderstanding with regard to this coefficient, the following illustration is offered with an apology to those who, being well acquainted with the matter, will no doubt regard it as superfluous:

Suppose that a number of bullets, all of the same size, be made, some of copper and some of gold; and that each copper bullet weighs one ounce.

Suppose further, in the first instance, that the specific gravity of gold be exactly double that of copper. Under these conditions, with any mixed lot of bullets—some of copper and some of gold—it is evident that the weight of the lot in ounces could at once be ascertained by counting the number of each kind separately, multiplying the gold number by two, and adding the result to the actual copper number. This represents the case for a single fixed coefficient.

Suppose now the conditions altered, and that the specific gravity of the gold bullets does not remain the same, but in a mixed lot is found to be higher when the gold bullets are relatively numerous, and lower when the reverse conditions prevail. To obtain the total weight for any mixed lot now, not only would the number of bullets of each metal have to be counted, but the ratio of the numbers would have to be determined, and the calculation be based on a specific gravity for gold which would vary with that ratio. This represents the case for a variable coefficient.

This will be apparent from the following example:

Case 1.—Conditions assumed: Moving Load, 9 tons; Fixed Load, 1 ton; Correct Coefficient, 2.0.

Here the correct equivalent Total Load would be $18 + 1 = 19$ tons. Had the coefficient been wrongly taken at 1.5, the equivalent Total Load would have worked out to $13.5 + 1 = 14.5$ tons, showing an error in the result nearly proportioned to the error in the coefficient, and all on the wrong side.

Case 2.—Conditions assumed: Moving Load, 1 ton; Fixed Load, 9 tons; Correct Coefficient, 1.3.

Here the correct equivalent Total Load would be $1.3 + 9 = 10.3$ tons. Had the coefficient been wrongly taken at 2.0, the equivalent Total Load would have worked out to $2 + 9 = 11$ tons, showing an error in the result very small as compared with the error in the coefficient, and all on the side of safety.

This system, with a fixed coefficient of 2.0, has found much favor in America, and has been adopted by several bridge engineers, notably by Theodore Cooper, M. Am. Soc. C. E. It has the advantage of very great simplicity, and gives results which show a very fair approximation to those obtained by experiment.

The corresponding formula is:

$$\text{Safe working stress in tons per square inch} = 7 \times \left(\frac{1}{1 + R}\right)$$

It will be observed on reference to Fig. 3, that the greatest percentage divergence from the standard curve occurs at the point where $\frac{\text{Fixed Load}}{\text{Moving Load}} = \frac{40}{60}$; here the results with a fixed coefficient of 2.0 would give an excess of strength of about 25 per cent.

In Fig. 2, for the coefficient, this system is represented by a straight line parallel to the datum line. In Fig. 3, for safe working stress, the bend of the curve is downward toward the datum line throughout, whereas the bend of the standard curve is in the opposite direction, with curvature increasing with the ratio of Moving Load. This peculiarity causes a considerable difference in the direction of the two curves where the ratio of Moving Load is high. The curve here in fact bends the wrong way.

Rule Adopted in India.—It will be observed that, where a fixed coefficient of 2.0 is adopted for all members of a truss, there will be an excess of strength for those members in which the Total Load is made

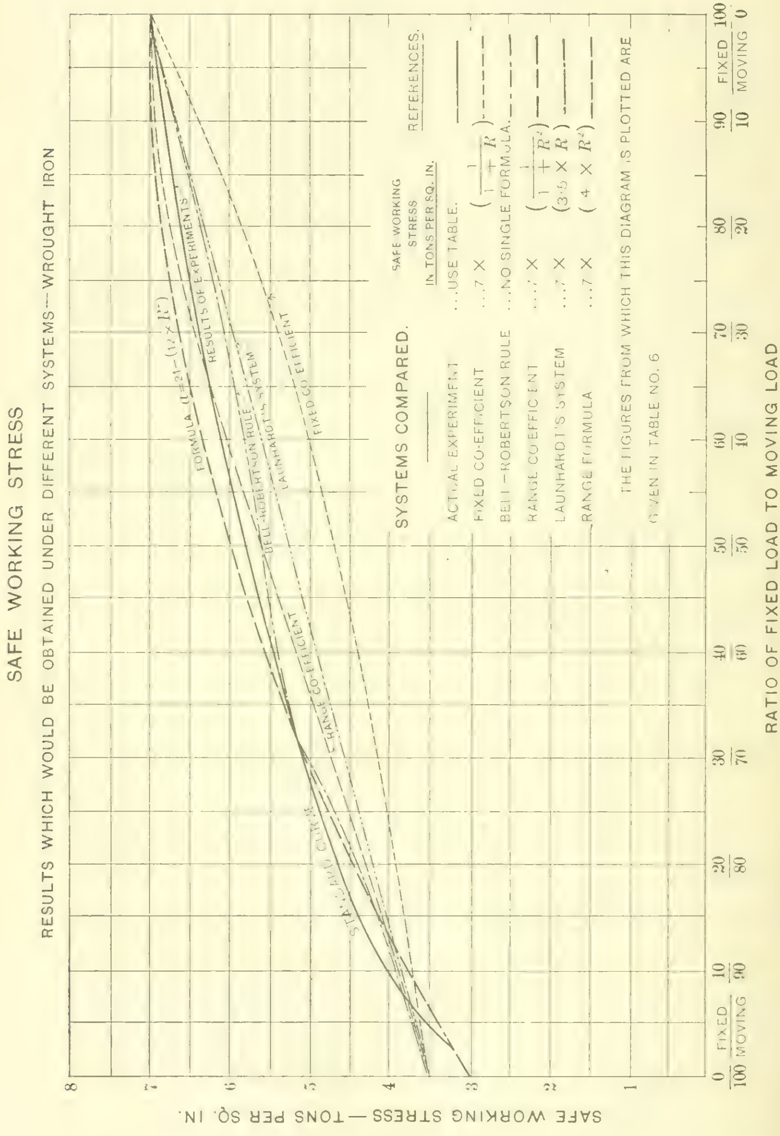
SAFE WORKING STRESS
RESULTS WHICH WOULD BE OBTAINED UNDER DIFFERENT SYSTEMS—WROUGHT IRON
SAFE WORKING STRESS — TONS PER SQ. IN.
RATIO OF FIXED LOAD TO MOVING LOAD
FIXED / MOVING
SYSTEMS COMPARED.
SAFE WORKING STRESS IN TONS PER SQ. IN.
REFERENCES.
FORMULA
RESULTS OF EXPERIMENTS
BELL-ROBERTSON RULE
LAUNHARDT'S SYSTEM
FIXED CO-EFFICIENT
RANGE CO-EFFICIENT
STANDARD CURVE
THE FIGURES FROM WHICH THIS DIAGRAM IS PLOTTED ARE GIVEN IN TABLE NO. 6

Fig. 3.

up of a large proportion of Fixed Load. This will be specially observable in the main booms of bridges of large span, for which a coefficient of 1.5 would be more suitable.

With a view to retain the simplicity of the fixed coefficient system, and, at the same time, obtain results more nearly in accord with actual conditions, the following rule has been adopted by the Government of India:

For any member of a railway bridge of wrought iron or steel, the total working load is to be taken as the greatest "Moving Load" multiplied by a coefficient and added to the actual "Fixed Load."

The coefficient to be used for this purpose is 2.0 in all cases, except for the upper and lower booms of triangulated girders, for which a coefficient of 1.5 may be used.

It will be seen that, although two coefficients are used, there will be no step or kink in the line representing the rule graphically. In the graphic representation of the rule there would be two separate lines, one for main booms and the other for general work.

Under the operation of the rule the coefficient of 1.5 would not be used for very small girders, inasmuch as in modern practice in India triangulated girders are not used for spans of less than 80 ft.

It has been shown that, with only a single coefficient, the adjustment of the unit stress, to suit varying ratios of Fixed Load to Moving Load, can be arranged for fairly well, provided the coefficient selected be that applicable to a condition of approximately "All Moving Load." With the addition of the second coefficient for main booms, the provisional rule adopted in India gives results which are probably as nearly in accord with the actual conditions as could be attained without the use of a variable coefficient. The rule has, moreover, the practical advantage of being simple and very easily applied.

This rule is practically in accord with that advocated by T. Claxton Fidler in his able and interesting exposition of his "Dynamic Method."* The formula given by Fidler is

$$\Omega = \text{Max. } S + \omega$$

where Ω represents the momentary internal stress and ω the dynamic increment. For cross-girders, vertical suspenders, diagonals of web bracing and girders up to a span of 20 ft.

$$\omega = \text{Max. } S - \text{Min. } S;$$

* "A Practical Treatise on Bridge Construction," by T. Claxton Fidler, M. Inst. C. E Edition 1893; p. 260.

this implies that the "effective stress" due to the Moving Load will be obtained by adding a "dynamic increment" equal to the Moving Load. In other words, the Moving Load is to be doubled to obtain its equivalent in terms of Fixed Load.

For main booms of girders of 100-ft. span or upward

$$\omega = \frac{\text{Max. } S - \text{Min. } S}{2};$$

this implies that the "effective stress" due to the Moving Load will be obtained by adding a "dynamic increment" equal to half the Moving Load. In other words, the Moving Load is to be increased by half as much again to obtain its equivalent in terms of Fixed Load.

Bell-Robertson Rule.—This rule was proposed by Messrs. J. R. Bell and F. E. Robertson as an improvement to the Government of India rule given above; the object being to obtain a simple means of varying the coefficient according to the nature of the load.

The rule is as follows:

For any member of a railway bridge of wrought iron or steel, the total "working load" is to be taken as the greatest "live load" multiplied by a coefficient of 1.5 and added to the actual "dead load," provided that a minimum dead load equal to half the live load shall be taken into calculation wherever the actual dead load is less.

Hence, under this rule, the coefficient would be 2.0 for the condition of All Moving Load, and would decrease as the relative amount of Moving Load became less, until at the point $\frac{\text{Moving Load} = 66.7}{\text{Fixed Load} = 33.3}$, the coefficient would become 1.5. From this point onward there would be a fixed coefficient of 1.5.

This rule gives a good approximation to the results as obtained by experiment, and is at the same time simple and easy of application. It is, however, not a uniformly consistent rule, and, if represented graphically, will be seen to be made up of two curves of different characters, with their intersections at the point $\frac{\text{Moving Load} = 66.7}{\text{Fixed Load} = 33.3}$. There is thus a cusp or kink in the line at this point. (See Fig. 2.)

Range Coefficient.—With a coefficient increasing with the range of stress (directly as $1 + R$), a very good approximation is obtained. On this system it is assumed that the extra effect of the Moving Load, as compared with the same weight as Fixed Load, may, for practical pur-

poses, be taken as the nominal Moving Load multiplied by R. The coefficient to be applied to the Moving Load, to obtain its equivalent in terms of Fixed Load, will then be $1 + R$.*

The practical application of this rule is extremely simple, as the coefficient for any compound load is obtained at once by merely adding 1.0 to the decimal giving the proportion of Moving Load to Total Load.

Example:

Moving Load .9 of Total Load.
Then coefficient = 1.9.

Moving Load .66 of Total Load.
Then coefficient = 1.66.

The corresponding formula is:

$$\text{Safe working stress in tons per square inch} = 7 \times \left(\frac{1}{1 + R^2}\right)$$

In Fig. 2, for the coefficient, this system is represented by a straight line drawn from the point 2.0 for "All Moving Load," to 1.0 for "All Fixed Load."

In Fig. 3, for safe working stress, the curve representing this system has a double (or reversed) curvature. It will be seen that it gives a very good approximation to the standard curve throughout.

Launhardt's System.—Launhardt's formula is:

$$a = u\left(1 + \frac{t - u}{u} \cdot \frac{\text{Min. } S}{\text{Max. } S}\right)$$

Avoiding troublesome fractions, the nearest approximation to the results for total effect, obtained by experiment, will be found by assigning to u a value of one half t. For wrought iron, t being taken as 21 tons, the formula would then become—

$$\text{Breaking stress in tons per square inch} = 10.5 \times (1 + \phi);$$

and with a factor of safety of 3.0—

$$\text{Safe working stress in tons per square inch} = 3.5 \times (1 + \phi)$$

It is to be noted that this result is the same as that which has been arrived at in America by an entirely independent course of reasoning, and is now adopted in some of the best modern American specifications.

Using the symbol R instead of ϕ, the formula would be written thus:

$$\begin{aligned}\text{Safe Working Stress} &= 3.5\,(2 - R)\\ &= 7.0 - 3.5\,R.\end{aligned}$$

---

* As a peculiarity incidental to this system, it may be noted that for any Compound Load in which Fixed Load + Moving Load = 100, the equivalent (or effective) Total Load will be $100 + (10\,R)^2$. For example, $\frac{\text{Fixed Load} = 20}{\text{Moving Load} = 80}$, then $R = .8$, and the effective Total Load is 164.

The use of this formula, with the values adopted, involves the following assumptions:

(*a*) The coefficient to be applied to the Moving Load, to obtain its equivalent in terms of Fixed Load, is 2.0 for the condition of "All Moving Load," diminishing as the ratio of Moving Load to Fixed Load decreases, until it becomes 1.5 for the condition of "All Fixed Load." The variation in the value of the coefficient is directly in the ratio of Total Load to Fixed plus Total Load. (See Fig. 2.)

(*b*) The nominal breaking stress varies directly as the ratio of Fixed Load to Total Load. Hence, in Fig. 3, for "nominal" safe working stress, this system is represented by a straight line.

This formula is very simple in application and has the advantage of being already well known and established in practice in America. It will also be seen by a reference to Fig. 3 that the results obtained on this system give a fairly good approximation to the results obtained by experiment.

It will be observed, however, that, being a straight line formula (see Fig. 3), it is impossible by any adjustment of the relative values of u and t to obtain results for safe working stress decreasing in a higher ratio as the percentage of Moving Load becomes greater. To give such results, and thereby be more in accord with the facts as indicated by experiment, the line as plotted in Fig. 3 should bend upward, *i. e.*, be convex as viewed from the top of the diagram.

Range Formula.—It being established that the breaking stress for a bar becomes less as the range of stress increases, it is evident that a useful formula may be constructed under which the breaking stress for a compound load may be arrived at by means of an expression which will show directly, as a function of the range, the amount by which the static breaking stress is to be reduced. On the system now proposed, therefore, the static breaking stress will be the standard for comparison in all cases, and the lower breaking stress for any compound load will be ascertained by a direct subtraction of the amount by which the breaking stress is reduced.

Avoiding troublesome fractions and complicated expressions, it is considered that for wrought iron, with a static breaking stress of 21 tons per square inch, the most suitable formula for general use would be:

Breaking stress, in tons per square inch, $= 21 - (12 \times R^2)$

and with a factor of safety of 3.0:

Safe working stress, in tons per square inch, $= 7 - (4 \times R^2)$

In Fig. 2, for the coefficient, the results obtained by the use of this formula are represented by a line extending from 2.33 for "All Moving Load," to 1.00 for "All Fixed Load."

In Fig. 3, for safe working stress, it will be observed that for the lower ratios of Moving Load, the working stress which would be allowed under this formula is somewhat higher than that given by the standard curve. The greatest percentage of difference is at the point where the Moving Load is about half the Fixed Load — here, under the formula, the permissible working stress per square inch would be 6.56 tons, as against 6.28 tons indicated as suitable by the results of experiment. The line represented by this formula crosses the standard curve near the point where the Moving Load is about double the Fixed Load; and thence for the higher ratios of Moving Load the working stress which would be allowed under the formula is lower than that indicated by the experimental results. At the point where the compound load becomes "All Moving Load," the permissible working stress, as given by the formula, is 3.0 tons per square inch.

General Summary.—The six systems, which have now been compared and discussed, for giving approximately the safe working stress for wrought iron, are:*

(*a*) *Actual Experimental Results:*

Safe working stress obtained by use of table.

(*b*) *Fixed Coefficient:*

Safe working stress in tons per square inch $= 7 \times \left(\frac{1}{1 + R}\right)$

(*c*) *Bell-Robertson Rule:*

No single formula applicable.

(*d*) *Range Coefficient:*

Safe working stress in tons per square inch $= 7 \times \left(\frac{1}{1 + R^2}\right)$

(*e*) *Launhardt's System:*

Safe working stress in tons per square inch $= 7 - (3.5 \times R)$

(*f*) *Range Formula:*

Safe working stress in tons per square inch $= 7 - (4 \times R^2)$

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* In these formulas the symbol R represents the proportionate range of stress, thus:

$$R = \frac{\text{Moving Load}}{\text{Fixed Load} + \text{Moving Load}} = \frac{\text{Range}}{\text{Total}}$$

The "Safe Working Stress" in each case is "Nominal Stress" due to the weight of the Moving Load and Fixed Load simply added. The corresponding "Effective Stress" obtained by the use of these formulas is 7 tons per square inch throughout.

TABLE No. 5.—RESULTS FOR THE COEFFICIENT. WROUGHT IRON.

Nature of Compound Load. Ratio Percentage. Fixed Load.	Moving Load.	Effect of Moving Load—Percentage. Compared with the Effect Due to the Same Weight Applied as Fixed Load. Results of Experiment for Actual Total Effect.	Fixed Coefficient. 2.0 $a = 21 \times \left(\frac{1}{1 + R}\right)$	Bell-Robertson Rule.	Range Coefficient. $a = 21 \left(\frac{1}{1 + R^2}\right)$	Launhardt's System $u = \frac{1}{2} t$. $a = 21 \left(10.5 \times R\right)$	Range Formula. $a = 21 \left(12 \times R^2\right)$
0	100	....	200.00	200.00	200.00	200.00	233.33
2.5	97.5	220.31	200.00	197.44	197.50	197.56	221.96
5	95	199.04	200.00	194.74	195.00	195.24	212.09
10	90	180.68	200.00	188.89	190.00	190.91	195.74
15	85	170.42	200.00	182.35	185.00	186.96	182.73
20	80	163.16	200.00	175.00	180.00	183.33	172.07
25	75	157.44	200.00	166.67	175.00	180.00	163.16
30	70	152.75	200.00	157.14	170.00	176.92	155.56
33.3	66.7	150.30	200.00	150.00	166.67	175.00	151.06
35	65	149.07	200.00	150.00	165.00	174.07	148.96
40	60	145.93	200.00	150.00	160.00	171.43	143.17
45	55	143.31	200.00	150.00	155.00	168.96	138.00
50	50	140.90	200.00	150.00	150.00	166.67	133.33
55	45	138.70	200.00	150.00	145.00	164.51	129.08
60	40	136.63	200.00	150.00	140.00	162.50	125.16
65	35	134.71	200.00	150.00	135.00	160.60	121.51
66.7	33.3	134.13	200.00	150.00	133.33	160.00	120.34
70	30	132.94	200.00	150.00	130.00	158.82	118.07
75	25	131.28	200.00	150.00	125.00	157.14	114.81
80	20	129.73	200.00	150.00	120.00	155.56	111.70
85	15	128.36	200.00	150.00	115.00	154.05	108.68
90	10	127.13	200.00	150.00	110.00	152.63	105.75
95	5	126.02	200.00	150.00	105.00	151.27	102.86
100	0	125.00	200.00	150.00	100.00	150.00	100.00

It will be observed that, of these systems, each has some advantage to recommend it, and that in the selection of the most suitable system for general use, it is to some extent a question of how far it may be considered advisable to sacrifice simplicity to obtain a closer approximation to the results obtained by experiment.

For convenience of comparison the results which would be obtained by the use of each rule or formula for different ratios of Fixed Load to Moving Load are exhibited in the following tables:

Table No. 5.—Results for the Coefficient.

Table No. 6.—Results for Safe Working Stress.

Table No. 7.—Percentage Comparison.

From the latter table can be ascertained at a glance the practical effect of any rule or formula on the area in cross-section of a member as compared with the area which would be indicated as the result of experiment.

TABLE No. 6.—RESULTS FOR SAFE WORKING STRESS. WROUGHT IRON.

Nature of Compound Load. Ratio Percentage.		Safe Working Stress with Factor of Safety = 3.0. Tons per Square Inch.					
Fixed Load.	Moving Load.	Results of Experiment for Actual Total Effect.	Fixed Coefficient 2.0 $a = 21 \times \left(\frac{1}{1+R}\right)$	Bell-Robertson Rule.	Range Coefficient. $a = 21 \times \left(\frac{1}{1+R^2}\right)$	Launhardt's System. $u = \frac{1}{2} t$ $a = 21$ $(10.5 \times R)$	Range Formula. $a = 21$ $(12 \times R^2)$
0	100	....	3.50	3.50	3.50	3.50	3.00
2.5	97.5	3.22	3.54	3.59	3.59	3.59	3.20
5	95	3.61	3.59	3.68	3.68	3.67	3.39
10	90	4.06	3.68	3.89	3.87	3.85	3.76
15	85	4.38	3.78	4.12	4.06	4.02	4.11
20	80	4.65	3.89	4.38	4.27	4.20	4.44
25	75	4.89	4.00	4.67	4.48	4.37	4.75
30	70	5.11	4.12	5.00	4.70	4.55	5.04
33.3	66.7	5.24	4.20	5.25	4.85	4.67	5.22
35	65	5.31	4.24	5.28	4.92	4.72	5.31
40	60	5.49	4.37	5.38	5.15	4.90	5.56
45	55	5.65	4.52	5.49	5.37	5.07	5.79
50	50	5.81	4.67	5.60	5.60	5.25	6.00
55	45	5.96	4.83	5.71	5.82	5.42	6.19
60	40	6.11	5.00	5.83	6.03	5.60	6.36
65	35	6.24	5.18	5.96	6.24	5.77	6.51
66.7	33.3	6.28	5.25	6.00	6.30	5.83	6.56
70	30	6.37	5.38	6.09	6.42	5.95	6.64
75	25	6.49	5.60	6.22	6.59	6.12	6.75
80	20	6.61	5.83	6.36	6.73	6.30	6.84
85	15	6.71	6.09	6.51	6.85	6.47	6.91
90	10	6.82	6.36	6.67	6.93	6.65	6.96
95	5	6.91	6.67	6.83	6.98	6.82	6.99
100	0	7.00	7.00	7.00	7.00	7.00	7.00

The results are also exhibited graphically for the coefficient in Fig. 2, and for safe working stress in Fig. 3.

With reference to the remarks made on this subject when discussing the fixed coefficient system, it will be observed that where the ratio of Fixed Load, as compared with Moving Load, is high, very great differences in the coefficient, as given in Table No. 5, have but a small effect on the practical results, as given in Tables Nos. 6 and 7.

Approximate Nature of Results.—It is to be remembered that the figures here quoted as "determined by experiment" are merely the averages of a large number of results, among which there are great discrepancies; and that for each point determined the maximum and minimum often differ widely from each other. Further, that in deciding where to place the average value, various circumstances must be taken into account and allowed due weight; but the precise amount of correction required must in each case (within certain limits) remain

TABLE No. 7.—PERCENTAGE COMPARISON. WROUGHT IRON.

Nature of Compound Load. Ratio Percentage.		Area of Members in Cross-Section. Compared with the Area Indicated by Experiment. Percentage.					
Fixed Load.	Moving Load.	Results of Experiment for Actual Total Effect.	Fixed coefficient 2.0 $a = 21 \times \left(\frac{1}{1+R}\right)$	Bell-Robertson Rule.	Range Coefficient. $a = 21 \cdot \left(\frac{1}{1+R^2}\right)$	Launhardt's System. $u = \frac{1}{2}t$ $a = 21 - (10.5 \times R)$	Range Formula. $a = 21 - (12 \times R^2)$
0	100	....	....	....	....	....	....
2.5	97.5	100	90.89	89.74	89.76	89.79	100.74
5	95	100	100.48	97.89	98.03	98.14	106.39
10	90	100	110.08	104.28	104.86	105.33	107.85
15	85	100	115.74	106.35	107.74	108.79	106.54
20	80	100	119.57	106.29	108.94	110.72	104.74
25	75	100	122.31	104.84	109.21	111.83	103.00
30	70	100	124.16	102.25	108.83	112.36	101.43
33.3	66.7	100	124.81	99.87	108.17	112.33	100.38
35	65	100	125.09	100.46	107.85	112.32	99.95
40	60	100	125.43	101.91	106.62	111.99	98.70
45	55	100	125.17	102.97	105.19	111.40	97.64
50	50	100	124.53	103.78	103.78	110.70	96.86
55	45	100	123.49	104.33	102.42	109.89	96.31
60	40	100	122.11	104.66	101.17	109.02	96.00
65	35	100	120.38	104.77	100.09	108.08	95.88
66.7	33.3	100	119.71	104.78	99.76	107.74	95.87
70	30	100	118.31	104.66	99.20	107.07	95.94
75	25	100	115.93	104.34	98.54	106.00	96.18
80	20	100	113.26	103.83	98.17	104.87	96.60
85	15	100	110.30	103.11	98.08	103.70	97.17
90	10	100	107.09	102.23	98.33	102.48	97.92
95	5	100	103.65	101.18	98.96	101.25	98.86
100	0	100	100.00	100.00	100.00	100.00	100.00

a matter for judgment and discretion. In the interpretation of the results of Wöhler's experiments, for example, it will be seen from Tables Nos. 2 and 3 that such eminent authorities as Gerber and Launhardt are by no means in accord as to the value of the coefficient where the ratio of Fixed Load is high, the coefficient for wrought iron for the condition of "All Fixed Load," according to Launhardt, being one-seventh greater than that obtained by Gerber.

On this subject Weyrauch remarks as follows : *

"Considering that absolutely exact laws for constructive materials will certainly never result from experiments, that even in brands of iron acknowledged to be good, differences in the statical breaking strength t of as much as 40% occur, and that it is merely a question of finding a substitute for the still more rough and incorrect assumption of a constant a, even the preceding might suffice for practical purposes until more facts are accumulated."

* *Proc. Inst. C. E.*, 1880-81, Vol. lxiii, pp. 282 and 283.

"Even for more exact determinations than those under consideration such an approximation ought to be considered satisfactory. To the author's mind it would appear sufficient if the deviations of the real values of a from those given by the formula did not exceed the deviations from one another of real values of a in good and commonly used materials."

If, therefore, by means of a simple rule or easily applied formula a fairly good approximation be obtained to the figures representing the result of experiment, it will evidently not be worth while to adopt a more complicated or troublesome system in order to secure a very close approximation to a line, the correct position of which (within certain limits) is after all a matter of some uncertainty.

The Selection of a System.—In the immediate effect, as determined by experiments, it is to be noted that the only effects allowed for are those which produce a measurable elongation or shortening of a bar or deflection of a beam, and the effect of the violent concussion or vibration, with which the application of the load is accompanied, is not taken into account.

It appears reasonable, however, to suppose that where the application of a load is accompanied by violent jarring or shock, there must be an effect on the bar more severe than that indicated by the mere temporary elongation, shortening or deflection.* The effect of jarring and vibration would be most severe with members (such as cross-girders and rail bearers) which are exposed more immediately to the action of the locomotive, and generally with members for which the ratio of Moving Load is high as compared with Fixed Load. Hence, in making allowance for the effect of shocks and violent jarring and vibration, it will be proper to arrange that the allowance shall increase as the ratio of Moving Load to Total Load becomes greater, or, in other words, as the value of R becomes higher.

* In this connection the following illustrations are offered for consideration :—

Let two precisely similar bars be subjected to repeated deflections, equal in extent, involving stress beyond the elastic limit. Let the deflections of the first bar be produced by steady pressure applied and removed. Let the deflections (equal in extent) of the second bar be produced by successive blows of a hammer. It would be expected that the number of deflections before breaking would be less in the second case than in the first.

Again—Let a glass tube be supported in a vertical position, and closed at the lower end. Into this tube let a certain quantity of dry angular sand, not sufficient to fill the tube, be poured in quietly. Having marked on the tube the height occupied by the sand, let the sand be emptied out and again poured in as before. This second time, however, let the tube be violently jarred and shaken as the sand falls into it. In the second case, the sand will not stand at so great a height in the tube as in the first case.

Again—Let a bar of steel be placed with its axis pointing to the magnetic pole, and be left undisturbed. In the course of a few years it will have become to some extent magnetic. The same bar in that position if violently jarred by the blows of a hammer would have become equally magnetic in the course of a few minutes.

TABLE No. 8.—RESULTS FOR WROUGHT IRON. OBTAINED BY USE OF RANGE FORMULA. BREAKING STRESS = $21 - (12 \times R^2)$.

Nature of Compound Load. Ratio Percentage.		Breaking Stress for the Compound (nominal) Load. Tons per Square Inch.			Coefficient to obtain equivalent of Moving Load in terms of Fixed Load.	Permissible Working Stress. Factor of Safety 3.	
			Apportionment of the Breaking Stress.				
Fixed Load.	Moving Load.	For the Total Load.	Due to Fixed Load.	Due to Moving Load.		Nominal Stress Tons per Square Inch.	Effective Stress Tons per Square Inch.
0	100	9.0000	0.0000	9.0000	2.3333	3.0000	7.00
2.5	97.5	9.5928	0.2398	9.3530	2.2196	3.1976	7.00
5	95	10.1700	0.5085	9.6615	2.1209	3.3900	7.00
10	90	11.2800	1.1280	10.1520	1.9574	3.7600	7.00
15	85	12.3300	1.8495	10.4805	1.8273	4.1100	7.00
20	80	13.3200	2.6640	10.6560	1.7207	4.4400	7.00
25	75	14.2500	3.5625	10.6875	1.6316	4.7500	7.00
30	70	15.1200	4.5360	10.5840	1.5556	5.0400	7.00
33.3	66.7	15.6672	5.2224	10.4448	1.5106	5.2224	7.00
35	65	15.9300	5.5755	10.3545	1.4896	5.3100	7.00
40	60	16.6800	6.6720	10.0080	1.4317	5.5600	7.00
45	55	17.3700	7.8165	9.5535	1.3800	5.7900	7.00
50	50	18.0000	9.0000	9.0000	1.3333	6.0000	7.00
55	45	18.5700	10.2135	8.3565	1.2908	6.1900	7.00
60	40	19.0800	11.4480	7.6320	1.2516	6.3600	7.00
65	35	19.5300	12.6945	6.8355	1.2151	6.5100	7.00
66.7	33.3	19.6668	13.1112	6.5556	1.2034	6.5556	7.00
70	30	19.9200	13.9440	5.9760	1.1807	6.6400	7.00
75	25	20.2500	15.1875	5.0625	1.1481	6.7500	7.00
80	20	20.5200	16.4160	4.1040	1.1170	6.8400	7.00
85	15	20.7300	17.6205	3.1095	1.0868	6.9100	7.00
90	10	20.8800	18.7920	2.0880	1.0575	6.9600	7.00
95	5	20.9700	19.9215	1.0485	1.0286	6.9900	7.00
100	0	21.0000	21.0000	0.0000	1.0000	7.0000	7.00

On the other hand, of the irregular or abnormal effects to be covered by the factor of safety, it will be observed that many of these causes of extra unit stress would have but little effect on a large and massive member on which the ratio of Fixed Load is high as compared with Moving Load; and it would therefore appear proper to allow a somewhat higher effective stress* as the ratio of Moving Load to Total Load becomes less, or, in other words, as the value of R becomes lower.

Range Formula Recommended.—These results are attained for wrought iron by the use of the range formula:

Safe working stress in tons per square inch = $7 - (4 \times R^2)$.

* Effective stress means the actual working stress to which the material is subjected, on the assumption that the coefficient adopted correctly represents the real effects of Moving Load as compared with that of the same weight as Fixed Load. In other words, "Effective Stress" means the stress due to the Fixed Load added to that due to the equivalent of the Moving Load in terms of Fixed Load.

With this formula it will be seen, by an inspection of Table No. 6 and of Fig. 3, that, as compared with the actual results of experiment, a somewhat higher effective stress would be allowed on the more massive parts of a large bridge truss where the irregular and abnormal effects would be least, and where economy of material may most profitably be exercised. On the other hand, with the lighter members of a truss (where the ratio of Moving Load is high as compared with Fixed Load, and where the irregular and abnormal effects would be most severely felt), the effective stress allowed would be somewhat lower than that deduced from the results of experiment.

The general results for wrought iron, obtained by the use of this formula, are exhibited in Table No. 8.

Results for Steel.

It will be observed that the value of u, as determined by Wöhler's experiments for steel, is but little greater than $\frac{1}{2}t$, instead of $\frac{2}{3}t$, the value obtained for wrought iron. The result is that the corresponding breaking stress, in the case of "All Moving Load," as deduced from these experiments, would be but little greater for steel than for wrought iron, although in the case "All Fixed Load" the breaking stress per unit of area for steel is one-third higher. This result would be somewhat unsatisfactory to bridge engineers, as, if acted upon, it would have the effect of requiring the area of a member subjected to a high ratio of moving load to be nearly as great with steel as with iron.*

It is to be noted, however, that much of the steel used in Wöhler's experiments was made about forty years ago.† The elastic limit of steel as now commonly used in bridge-building is probably higher, and there is no difficulty in obtaining a material having an elastic limit as high as two thirds the ultimate static breaking stress.

Hence it would appear that for the quality of mild steel, as now used for the construction of railway bridges, if the ultimate breaking stress per square inch be taken as 27 tons, the elastic limit may, for practical purposes, be taken as high as 18 tons. The rule for safe

* If the immediate effect of Moving Load be neglected, it would appear from the results of Wöhler's experiments that, under the condition of "All Moving Load," the breaking stress for steel might actually be less than that deduced for wrought iron, thus:

For wrought iron.............................. $u = \frac{2}{3}t$ and $t = 21$
For steel.. $u = \frac{1}{2}t$ and $t = 27$

The breaking stress for "All Moving Load" corresponding to these conditions would be, per square inch:

Wrought iron.. 14.00 tons
Steel.. 13.50 tons

† Wöhler's experiments were carried on for twelve years, from 1859 to 1870.

working stress may then be based on a value of u equal to $\frac{2}{3}t$* and the rule for steel may then take the same general form as that found suitable for wrought iron.

On these considerations the formula recommended for steel is:

Safe working stress in tons per square inch $= 9 - (5 \times R^2)$ corresponding with that proposed for wrought iron.†

The general results for steel, obtained by the use of this formula, are exhibited in Table No. 9.

TABLE No. 9.—RESULTS FOR STEEL. OBTAINED BY USE OF RANGE FORMULA. BREAKING STRESS $= 27 - (15 \times R^2)$.

Nature of Compound Load. Ratio Percentage.		Breaking Stress for the Compound (Nominal) Load. Tons per Square Inch.			Coefficient to Obtain Equivalent of Moving Load in Terms of Fixed Load.	Permissible Working Stress. Factor of Safety = 3.	
		For the Total Load.	Apportionment of the Breaking Stress.				
Fixed Load.	Moving Load.		Due to Fixed Load.	Due to Moving Load.		Nominal Stress. Tons per Square Inch.	Effective Stress. Tons per Square Inch.
0	100	12.0000	0.0000	12.0000	2.2500	4.0000	9.00
2.5	97.5	12.7410	0.3185	12.4225	2.1478	4.2470	9.00
5	95	13.4625	0.6731	12.7894	2.0585	4.4875	9.00
10	90	14.8500	1.4850	13.3650	1.9091	4.9500	9.00
15	85	16.1625	2.4244	13.7381	1.7889	5.3875	9.00
20	80	17.4000	3.4800	13.9200	1.6897	5.8000	9.00
25	75	18.5625	4.6406	13.9219	1.6061	6.1875	9.00
30	70	19.6500	5.8950	13.7550	1.5344	6.5500	9.00
33.3	66.7	20.3333	6.7778	13.5555	1.4918	6.7778	9.00
35	65	20.6625	7.2319	13.4306	1.4719	6.8875	9.00
40	60	21.6000	8.6400	12.9600	1.4167	7.2000	9.00
45	55	22.4625	10.1081	12.3544	1·3673	7.4875	9.00
50	50	23.2500	11.6250	11.6250	1.3226	7.7500	9.00
55	45	23.9625	13.1794	10.7831	1.2817	7.9875	9.00
60	40	24.6000	14.7600	9.8400	1.2439	8.2000	9.00
65	35	25.1625	16.3556	8.8069	1.2086	8.3875	9.00
66.7	33.3	25.3333	16.8889	8.4444	1.1974	8.4444	9.00
70	30	25.6500	17.9550	7.6950	1.1754	8.5500	9.00
75	25	26.0625	19.5469	6.5156	1.1439	8.6875	9.00
80	20	26.4000	21.1200	5.2800	1.1136	8.8000	9.00
85	15	26.6625	22.6631	3.9994	1.0844	8.8875	9.00
90	10	26.8500	24.1650	2.6850	1.0559	8.9500	9.00
95	5	26.9625	25.6144	1.3481	1.0278	8.9875	9.00
100	0	27.0000	27.0000	0.0000	1.0000	9.0000	9.00

* For steel having a higher static breaking stress per square inch than 27 tons, the elastic limit would no doubt bear a lower ratio to the ultimate breaking stress than 2 : 3. But in such case the elastic limit would merely be lower relatively, not lower absolutely. A rule based on a breaking stress per square inch, of 18 tons for "All Moving Load," ranging to 27 tons for "All Fixed Load," would clearly not be less safe for a material, the breaking stress for which might range up to (say) 36 tons for "All Fixed Load." Some of the steel used in Wöhler's experiments showed a static breaking stress of more than 50 tons per square inch.

† In this formula the symbol R represents the proportionate range of stress, thus:

$$R = \frac{\text{Moving Load}}{\text{Fixed Load} + \text{Moving Load}} = \frac{\text{Range}}{\text{Total.}}$$

1898. MAY. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

MARINE WOOD-BORERS.

By CHARLES H. SNOW, M. Am. Soc. C. E.

TO BE PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

The forms of marine life which attack wood are much more numerous than is commonly supposed. The wood-borer is for the most part concealed from view while at work, and is in consequence frequently disassociated from the subsequently discovered results of its labors.

The Teredo Navalis, being common in Europe, where these forms of life were first studied, has become better known than its companions. Nearly every kind of boring found in wood which has been in sea water is in consequence very generally attributed to this animal. The Teredo Navalis is worthy of all the attention it receives, being important of itself, and also standing as a representative of its family. That there are other species of teredo than the navalis, and other wood-borers than the teredo, must, however, not be forgotten.

The destruction accomplished by wood-borers is very great. The cost of wood destroyed, of replacements necessitated by failures, and of protecting wood so that it will not fail, aggregate a very large amount. The influence of these animals has not always been con-

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

fined to direct cost, and it is said that their presence in abnormal quantities has affected the prosperity of cities. The weakening of the pile foundations and other woodwork of the dikes of Holland once threatened the safety of that country.

The Teredo Navalis has been observed in Europe for nearly two centuries. The Dutch, from necessity, excelled in the early study of its habits, and the reputation thus established has continued to the present time. Much attention has been devoted to this and other forms of marine wood-borers by American naturalists, among whom the names of A. E. Verrill, of Yale University; S. I. Smith, of the National Fish Commission; W. H. Dall, of the Smithsonian Institution, and lately C. O. Siegerfoos, of the Johns Hopkins Department of Biology, should be mentioned.

The Teredo.

The teredo is a very ancient form of life, fossil remains having been found both in England and America. It was known to the Ancients, and is mentioned in the writings of Pliny and of Ovid. It was observed in modern times about the year 1730, when, as has been stated, it threatened the woodwork of the dikes of Holland.

The Teredo Navalis is often referred to as the Ship Worm, although it is a mollusk. The name is convenient, because the navalis and other teredos resemble worms in appearance, but if it is to be used it should include all forms of the teredo, and not be confined to the species navalis, in which case the names Teredo and Ship Worm rather than the names Teredo Navalis and Ship Worm would be synonymous.

Seven species of the teredo have been identified as existing in the United States. They are the Teredo Navalis (Linn.), the Teredo Norvegica (Spengler), the Teredo Dilatata (Simpson), the Teredo Megotara (Hanley), the Teredo Thompsono (Tryon), the Xylophaga Dorsalis (Forbes and Hanley), and the Xylotrya Fimbriata (Jeffreys). These varieties are similar in their principal characteristics. A description of the teredo may profitably be preceded by a mention of two of the more familiar forms of life which resemble it.

The long or soft shell clam (Myra), Plate XX, Fig. 1, is familiar to all. The prominent characteristic is a long worm-like neck, which is out of all proportion to the shell which covers the softer parts. This

FIG. 1

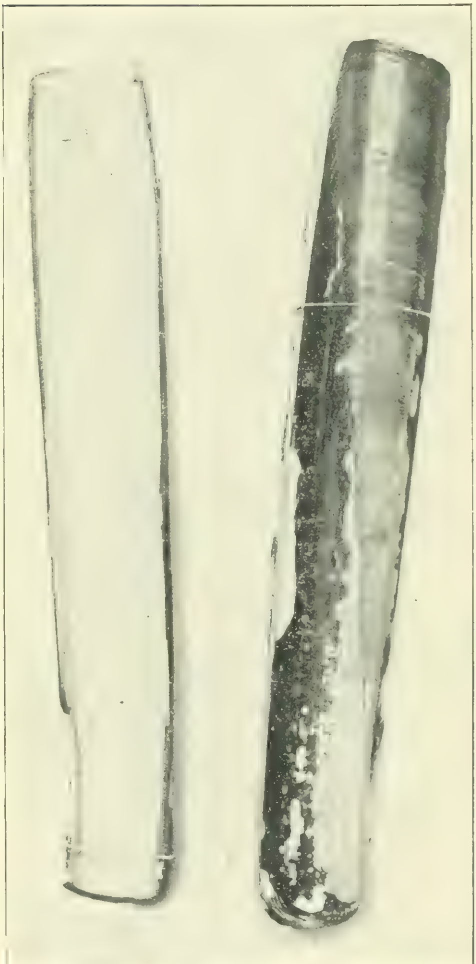

FIG. 2.

long neck contains two tubes or syphons, through one of which the animal receives its sustenance, the other being used to expel the water from which the microscopic life has been withdrawn.

Another form of the same worm-like structure is the razor clam (Solenidæ), Plate XX, Fig. 2, whose shape is more nearly conformed to by the covering shell. It possesses a powerful club-shaped foot or sucker, which is used as an auger to penetrate the sand in which it resides. The special features to which attention is called are the long worm-like syphon structure in both the long and razor clam, and the foot or sucker of the latter. A general idea of the teredo can be gained by imagining a soft clam with an unusually long neck and a very small shell, or a razor clam with its shell cut away so as to expose nearly all of its length. The long clam, the razor clam and the teredo are all true mollusks, and only resemble worms in that parts of their bodies are long and round. The shell of the adult teredo covers but a small proportion of its slender worm-like body.

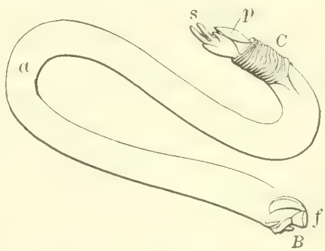

FIG. 1.

The body of the teredo (Fig. 1), which in substance resembles that of the oyster, is long, slender, smooth, soft and gray, tapering somewhat toward the outer or posterior end *s*, where it is marked by a wrinkled collar, shown at *C*. This is the end of the animal which is nearest to the entrance of the burrow. Although the collar attaches the animal to the sides of the boring which it has formed, yet the teredo is free to move to either side of it. The two little horns shown at *s* are the extremities of the tubes or syphons which have been described in connection with the long and razor clams. These little tubes may be extended out through the opening to the cell, and are the portion of the animal which is first noticed by the observer.

The two little shelly plates, *p*, at the base of the syphon prolongations, are called pallets. They are slightly curved at the top so that they can enfold the syphons. When the syphons are withdrawn into the burrow, the pallets are contracted over them so as to protect these soft tubes from the enemy. The pallet shells are frequently confused with the boring shells, which are at the other end of the body.

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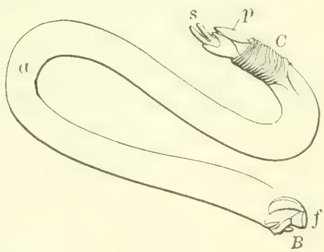

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The two little shelly plates, *p*, at the base of the syphon prolongations, are called pallets. They are slightly curved at the top so that they can enfold the syphons. When the syphons are withdrawn into the burrow, the pallets are contracted over them so as to protect these soft tubes from the enemy. The pallet shells are frequently confused with the boring shells, which are at the other end of the body.

The principal or boring shells *B*, which are open both at the top and bottom, are small, and very beautifully formed. The opening at the top permits the emergence of the body, and the foot or sucker *f* passes through the opening at the bottom. A calcareous substance is invariably deposited by the teredo upon the newly cut surface of the wooden tunnel, forming an enameled lining through which the animal can glide backward or forward as it contracts and expands. This shell tube is a separate structure, distinct from the pallets and from the boring shell.

It will be seen that the distinctive features of the teredo are the body, the collar, the syphons, the pallets, the boring shell, the foot and the lining shell. These several members and their processes will be considered separately.

The Body.—The body of the teredo resembles that of a long worm, without the articulations. In the young animal it is so transparent that some of the interior organs, such as the heart and the ovary, may be observed through it. The heart consists of two auricles and a ventricle. The pulsations, which may be readily counted, are irregular, the rate being about four or five per minute. The blood is a transparent, colorless fluid. Many of the important organs, as the mouth, the palpi, the liver and the foot, are enclosed in the boring shell at the further extremity of the animal. The gills are located for the most part at the outside of the shell, and are very interesting. They are long and narrow, usually reddish brown in color, and perform the important office of sheltering the eggs and embryo. The nervous system is well developed, and consists of filaments and ganglions connecting the mouth, the branchiæ, the foot, the collar and the syphons. The stomach is not distinguished by any peculiarity, but there is a well-developed intestine. The great length of the body is due to the elongation of the syphons or breathing tubes.

The Collar.—The collar is a muscular, wrinkled membrane which extends entirely around the posterior portion of the animal, and forms a connection between the teredo and the shelly lining of its tunnel. This is the only place at which the teredo is not free and separated from its surroundings. The collar fills the place between the teredo and the circumference of its tunnel. Water cannot pass through the orifice of the tunnel, save as it is controlled by the syphons. The collar contains several well-defined muscles, and these act upon the

FIG. 1.—LIFE SIZE.

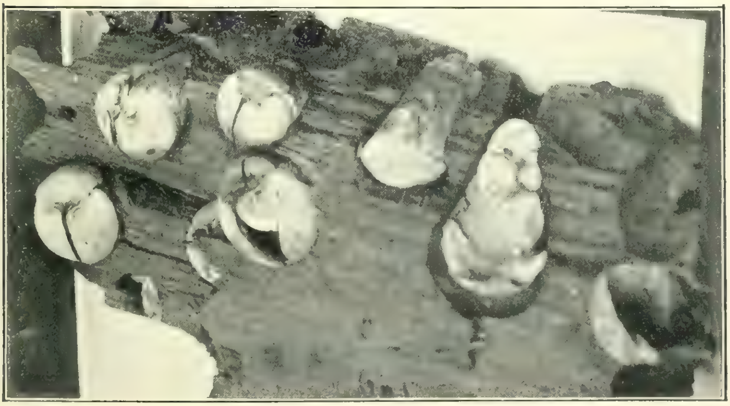


FIG. 2.—LIFE SIZE.

pallets which are pulled down over the syphons in such a manner as to close the entrance to the tube when the extremities of the syphons are drawn into the burrow.

The Syphons.—The syphons are the two principal organs, and extend throughout the greater length of the body. One of these tubes conveys the oxygen, water and infusorial food to the vital processes of the animal; the other conveys the exhausted water, the excretions, the débris from the excavation and the eggs to the free water without. The outer structures of the syphons are united while they remain in the body, but signs of divergence are seen as they emerge from between the pallets. They continue united for a little and are then separated into two distinct tubes, as indicated in Fig. 2, Plate XXII. These divergent extremities pass backward and forward through the orifice in the wood. They constitute the only part of the animal which can be seen from the outside of the wood. The extremities often stretch for some distance out through the minute opening to the cell, and are sometimes mistaken for the entire animal.

The extremities, which usually appear to be about equal in length as seen from the exterior of the wood, are yellowish or white in tint, but are sometimes speckled with reddish brown. The longer or incurrent extremity can be pushed out to a distance of 2 ins. or more, while the outcurrent throat remains at about half that distance.

The teredo is able to expand or contract these extremities at will and when the conditions are favorable, they are extended through the orifice to their full length, and remain stationary or wave slowly backward and forward. The motion is sometimes confined to the extreme ends, while the greater part of the extremity remains stationary. When the animal is alarmed, the syphons are withdrawn and pass down between the pallets into the tunnel; the pallets close over them and protect them from harm. The syphons are erected by means of a current of blood sent into them from the vessels within. When the water is warm, the animal is active and the syphons are extended out full length. They are withdrawn when the water becomes cold, and the teredo is entirely hidden. Fig. 1, Plate XXI, shows the syphons fully extended as they appear after several consecutive days of warm weather. The extremities of the syphons must always be kept at the orifice of the wood. As the animal grows, the

muscular collar and the pallets recede from the entrance, so as to permit the extremities of the syphons to remain there.

The Pallets.—The two shelly plates near the orifice are called pallets. They are broad and flattened at the top and concentrated at the base. These long slender basal columns are connected with the muscles of the collar so that the pallets may be relaxed or contracted at will. They relax when the syphon extremities pass out between the crescents or horns, which will be seen at the top of the shells, and contract when the syphons are withdrawn. The pallets are then folded over so as to serve as an operculum to protect the soft tubes from enemies. The Xylotrya Fimbriata differs from the ordinary variety in that its pallets are long and oar-shaped. The stalk is slender, the blade is flattened on the inside and is convex externally. It consists of ten or twelve funnel-shaped segt ments set into one another, and having their margins projecting at the sides, so tha the edges of the blade appear serrated.

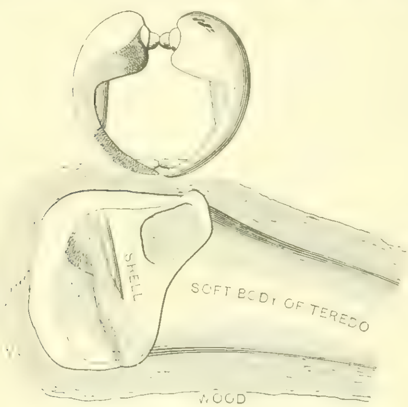


Fig. 2.

The Boring Shell.—The boring shell, which is nearly as long as it is broad, presents an irregularly triangular appearance when observed from the side. It may be best seen in Fig. 2. The two halves close tightly at the hinge and at the side opposite the hinge; the open space at the top being toward the main bulk of the animal, and the opening toward the extremity of the tunnel permitting the emergence of the foot or sucker. The shells of young animals are larger in proportion than those of old animals, as shown in Fig. 5, Plate XXII. When the animal is very young, it is for a short time entirely enclosed in the shell.

The Foot.—The foot which in form resembles a pestle is a short, stout, muscular organ, broadly truncated or rounded at the end, and so arranged that it can exert a powerful suction upon anything to which it is attached. This cupping action assists the shell in excavating to an extent which has probably not been understood.

FIG. 1.

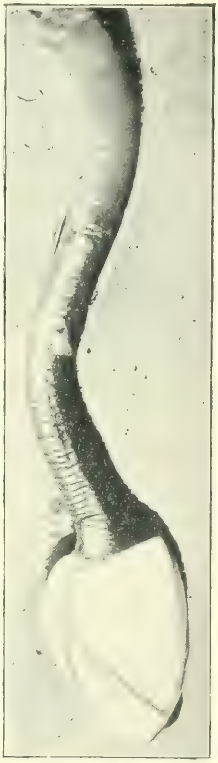


FIG. 3

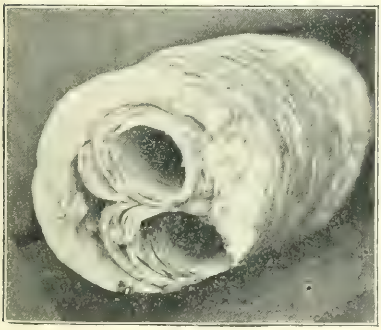


FIG. 2.

FIG. 5.

FIG. 4.

ALL LIFE SIZE EXCEPT FIG. 3.

The Lining Shell.—This shell, which has been previously described, follows the tunnel until it finally terminates in a spherical cap, which may be seen to the left of Fig. 1, Plate XXIII. The adult animal occasionally shrinks, and a second cap or partition may be formed as he retreats, the space between the caps remaining unoccupied.

The early portion of the tunnel, which was cut by the young animal and is now occupied by the syphon extremities, is below the normal diameter. The lining shell at this point sometimes divides itself into two tubes, each containing one of the syphons. This structure is well shown in Fig. 2, Plate XXII. It has been stated that the extremities of the syphons must be kept at the orifice of the wood. The collar and pallets recede as the animal grows larger, thus forming a considerable space between the pallets and the orifice of the tunnel, which is filled with little rings of shell, whose sharp edges serve to protect the entrance. The slender syphons emerging from the pallets pass readily through the spaces at the centers of the rings.

The teredo can rarely advance for any time in a straight line, being forced to deviate therefrom so as to pass around obstacles, such as cracks, knots or the pre-existing tunnels of its companions. The tunnels may wind in and out and pass so close to one another as to occupy almost the entire content. This fact is well illustrated in Fig. 2, Plate XXIII. The thickness of the lining varies with the species. It is sometimes so thin and fragile that it becomes detached by the slightest shock. Many of the dried specimens exhibited in museums do not show the lining shell for this reason. The shell is sometimes very thick. The specimen shown in Fig. 1, Plate XXII, exhibits an extreme thickness of half an inch. It was 41 ins. long and 3 ins. in diameter at the larger end. This species exists in the tropics, and does not penetrate wood, but forces its way into the sand, thus resembling the razor clam. The shells of this variety of teredo sometimes become covered with oysters or other mollusks. The lining presents a surface against which the soft body of the mollusk may press without injury. It seals the interior of the cell so that the water supply may be better controlled by the syphons.

The teredo rarely crosses a seam or joint in the wood, probably because it fears for the integrity of the lining. Specimens exhibiting the attempt of the teredo to cross a seam show that the shell has been much strengthened at the junction point. When the teredo arrives at

maturity, or whenever an insurmountable obstacle is encountered, it seals over the inner extremity of the lining. The growth of the animal, in length, is stopped in either case, and it is entirely surrounded by shell. There is a communication with the outer sea at the two syphon points only, and as the animal continues to live for some time under these conditions it is evident that its sustenance is derived from other sources than the wood.

Vital Processes.—The teredo resembles other bivalve mollusks in that it exists upon infusorial life. This food, together with the necessary amount of oxygen, is drawn in through the longer or incurrent syphon, and flows throughout the length of the animal until it reaches the mouth at the other extremity. The mouth, stomach and intestine are well developed and perform their usual offices. The oxygen is retained by the gills. The return current, beginning at the gills, removes the exhausted water, the excretions and the woody particles. These flow out through the animal and are ejected by the shorter or outcurrent syphon. The teredo does not devour wood; its form is such that dust and other débris have to pass through its body to the point of ejection. Where a teredo is watched for some time, small clouds of very fine dust may at length be observed puffed out from the orifice. The circulation through the syphons is continuous.

The teredo may live for a short time out of water. This fact explains its ability to attack wood between high and low water. The specimens which enter wood where it is exposed between the tides do not seem to be greatly hindered in their work.

Th. G. Hoech, M. Am. Soc. C. E., states that teredos have been found alive in wood which had been removed from the water for two months. The circumstances are not stated, but it is probable that the wood was in bulk and remained moist in consequence.

Many of the logs in a cargo of Central American woods recently received in New York City, after a voyage of about two weeks, were found to be occupied by living teredos, which had gained entrance to the wood while it was waiting shipment in Southern waters. They were alive, strong, apparently healthy and able to resume their work if the wood should be again submerged. It is probable that considerable water was contained in the wood and hold of the vessel, yet the logs were certainly not submerged, and the fact remains, that these particular teredos survived the voyage and were so numerous as to

FIG. 1.—LIFE SIZE.

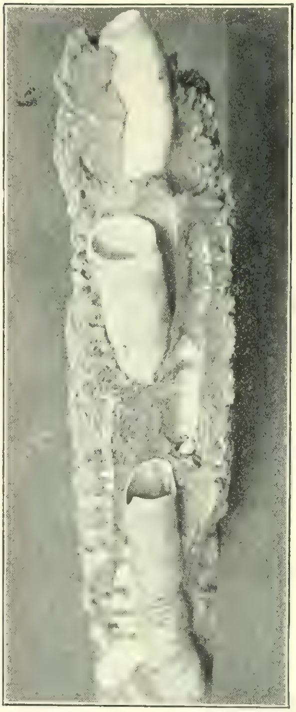


FIG. 3.—LIFE SIZE.

FIG. 2.—LIFE SIZE.

emit a strong, disagreeable, fishy odor, which necessitated the removal of the logs from the yard.

R. Montfort, M. Am. Soc. C. E., states that teredos occupying standing piles were killed when these piles were encircled by loose iron jackets.* The teredos were in no wise injured by the jackets and the presumption is that the jackets were forced into the mud and that the teredos were destroyed by the muddy water. Where infected vessels are subjected to fresh water, the teredos usually live for about two weeks, so far as known.

When specimens of wood containing teredos are removed from sea water, the syphons relax after the vitality of the animal is gone. If the specimen be held upside down, the syphons then fall outward to a distance of 1 in. or more, according to the size of the animal within.

The Boring Apparatus.—While the animal is still very small, it settles upon the surface of the wood and almost immediately begins to clear away a place in which to burrow. A small pit is made by the edges of the valves of the shell, which come together on pivots shown in Fig. 2. The shells are controlled by powerful muscles acting so as to swing them backward and forward upon the pivots. Only a few of the teeth upon the shell are shown in Fig. 2, and these are exaggerated in size. When the posterior muscle contracts, the shell, with the teeth, is thrown outward and backward and rasps upon the surfaces of the wood. The process is assisted by the foot which emerges through the large blank space between the shells and performs a cupping action.

The teredo differs from many of the pholos tribe. It probably employs the inflated syphons and pallets to some unknown extent as a fulcrum. The lower portion of the animal has a large field of rotation, so that the various portions of the anterior end of the burrow can be grated away in turn. The large end of the burrow is, of course, slightly larger than the diameter of the shell. The principal work of excavation is accomplished by the shell. Many of the stone-borers, on the other hand, brace themselves by means of the shell which they open against the walls of their burrow and hold there as a fulcrum. The muscular foot emerges from the shell, and, assisted by the grit of previous borings, grinds at the stone. The wood-borers, so far as known, excavate with the shell, assisted by the foot. Xylophaga and teredina probably bore like the teredos.

* *Transactions*, Vol. xxxi, p. 227.

The Character of the Excavation.—The teredo is very small when it begins to attack the wood, and the hole by which entrance is made, which is the only perforation that appears upon the exterior, is very minute (see Fig. 1, Plate XXIV). The animal develops very rapidly. The adult diameter is usually attained within 1 or 2 ins. of the surface, and the burrow increases in diameter regularly from the point of entrance to the maximum diameter. The animal grows principally in the direction of length, and therefore, it attacks the wood so as to accommodate its quarters to this increase in length. The boring is first carried on across the grain, but ordinarily turns within a short distance and passes in the direction of the grain. This general direction is usually followed, but obstacles are so frequently encountered that the tunnels become exceedingly tortuous, and pass in every conceivable direction.

The teredo usually passes around knots, although quite competent to penetrate knots of oak and other hard woods. Adjoining tunnels are not encroached upon, because these tunnels are completely occupied by live teredos, and more ingenuity would be required to pass through one of them than to avoid it. When cracks exist in the centers of large timbers, they are approached from all sides, but the film is never willingly broken through. It may be assumed that the teredo does not desire to cross any crack or sign of cleavage. It prefers wood that is not surrounded by bark, because of the line of contact between the wood and the bark. When a piece of wood is thoroughly infested, the animals have to pass very close to one another, and the thin film of wood left between the adjacent tunnels is reinforced by the calcareous lining. E. L. Corthell, M. Am. Soc. C. E., states that the teredos pass through the willows of the Mississippi improvements so as to leave them a mass of nearly parallel tubes. The calcareous lining appears to lend some degree of strength and toughness to the wood thus weakened.

A fact which is of great importance is that the teredo must always command the opening of its burrow, and have free and permanent access to the water. One end of the teredo being thus fastened at the outside of the wood, the depth of penetration must be limited to the length of the animal. It cannot exist in the interior of woodwork, nor can it live or breed without actual contact with free water. A report that a teredo had been found in the interior of one of the

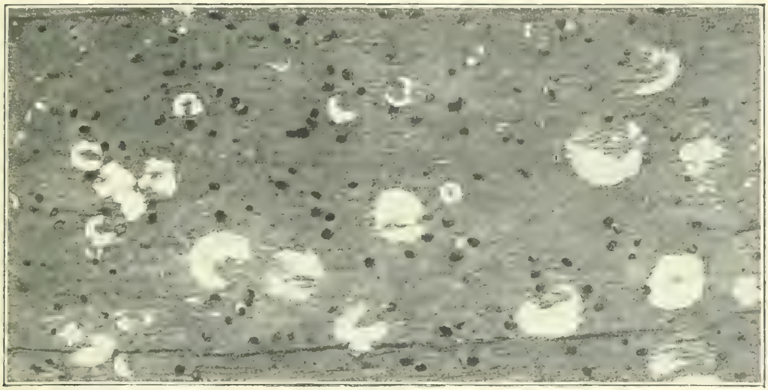


Fig. 1. Life Size.

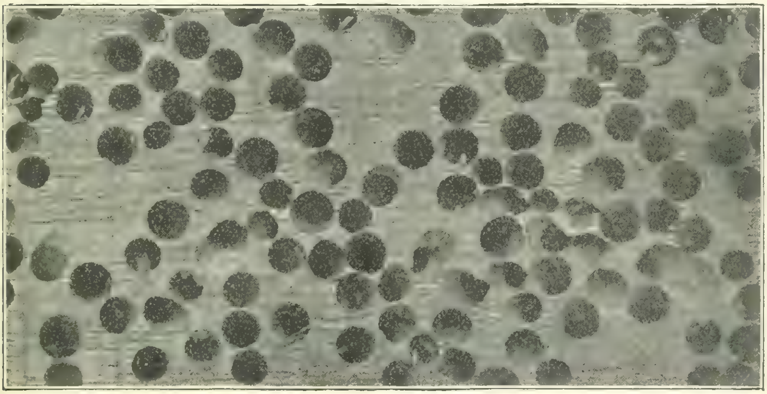


Fig. 2. Life Size.

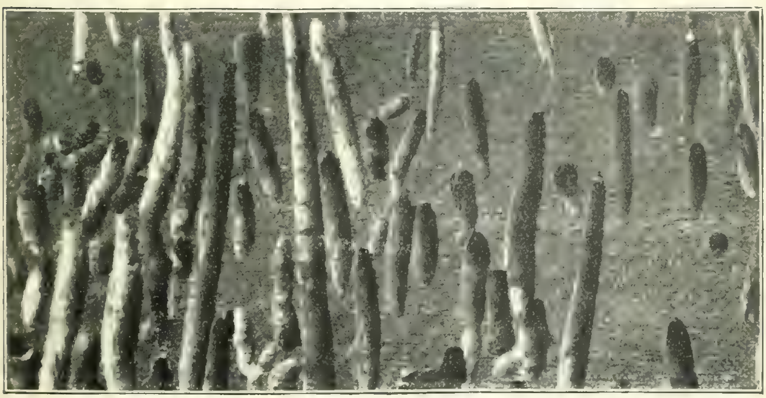


Fig. 3.—Life Size.

caissons of the Brooklyn Bridge was the cause of much popular and unwarranted excitement at the time of the erection of that structure.

It is impossible to examine the work of the teredo without surprise at the smooth surface of the burrow, which is cut as perfectly as by the sharpest chisel. In still weather the teredo may be plainly heard while at work. More than 50% of the weight of the wood may be removed by the teredo, without being greatly evidenced upon the surface. The little holes by which the animal gains entrance become partially obliterated when it dies. Wood may appear to be quite sound and yet be so weakened that much of it can be crushed by the hand (see Plate XXV). Failure, therefore, frequently comes suddenly. The tops of piles which appear to be in good condition are suddenly seen floating away. A large wharf at Provincetown, Cape Cod, unexpectedly collapsed by reason of the teredo. A freight train on the Louisville and Nashville Railroad crushed through a trestle which had been standing but ten months, and had been constantly inspected without showing signs of weakness. An examination showed that the piles in this instance had been eaten off close to the mud line.

Plate XXV is from a photograph of a log of Panama mahogany, which was cut in the uplands of the Isthmus and floated through fresh water to the harbor of Colon, where it remained floating in salt water while awaiting shipment. The log was overlooked for one season, and the work of the teredo is thought to have been accomplished in about nine months. The heavy, wet specimen was shipped with others, under the impression that it was sound, but it broke by its own weight after its arrival in New York.

The Size of the Teredo.—The size of the teredo depends upon the species, locality and age, and the absence of obstacles to excavation. Specimens of wood submitted for examination are frequently divested of any signs by which the excavators can be classified. The excavation can be identified as the work of the teredo, but the exact species accountable for the work cannot always be told.

Locality has much to do with development. Specimens grow more rapidly and attain larger size where the climate is warm. The teredo continues to grow until it reaches its maximum size, unless an obstacle is encountered. The species Navalis may be assumed to average from about one-fourth to three-eighths of an inch in diameter and from about 10 to 15 ins. in length, but specimens of the teredo frequently attain a

much greater size. Professor Siegerfoos writes that he has measured them up to 4 ft. in length and that the specimens thus measured had not arrived at their full limits. A log about $2\frac{1}{2}$ ft. square, recently examined, was found to be entirely honeycombed, the borings starting from opposite sides and passing slantingly into the wood, so that they probably averaged over 2 ft. in length. The specimen was of particular interest because the destruction had been accomplished in a single season.

The minimum diameter and length of a boring may be taken as $\frac{1}{4}$ in. and 5 ins. respectively. The maximum length may be taken as 4 ft. The largest diameter ever noticed by the author measured $1\frac{1}{8}$ ins.; this is shown on Plate XXVI. After the teredo has penetrated the wood for a little distance, the diameter remains about constant. Diameters are measured in this portion of the burrow and not at the entrance.

The Range or Field of Work.—The teredo operates throughout a vertical field of considerable depth. This field begins at a point a little above low-water mark, and extends downward until the pressure becomes too great, or the soil at the bottom is encountered. The teredo seems to be able to exist for a little time without submergence, and is therefore able to live above the low-water mark, although exposed daily between the tides. The interior extremity of the tunnel may be higher up than the entrance. The upper limit of the excavated wood cannot be determined by an examination of the orifices at the surface. The lower limit is uncertain and is probably different for different species. It has been assumed by many that the lower limit could be set at about 14 ft. below low water, but recent information, thought to be reliable, indicates that piles have been affected at a depth of from 20 to 25 ft. below that level. The fact that the interior extremity of the burrow is often found below the mud line has given the impression that the field of the animal may extend below this limit, but the outside opening or entrance made by the teredo is never below the soil, although the boring may turn downward for the whole length of the animal. If sediment accumulates around the bottom of the wood so as to cover the syphons, the death of the teredo results.

It is reported that in some harbors the teredos attack at the surface, and in others at the mud line. These differences are partially

LIFE SIZE.

due to differences in the constituents of the upper and lower layers of water. Where the fresh water of a river meets the heavy water of a sea, the teredo may be almost entirely confined to the lower stratum. The range or field of the teredo is important, because protective processes which could be confined to this field would be more economical than those in common use which are applied to the entire structure.

The Rapidity of the Work.—The rapidity of the work of the teredo depends upon conditions similar to those which govern its size. The evidence upon this subject is not always accompanied by a statement of the conditions under which the results were accomplished, such as the species of teredo, the character of the wood, the season, the climate and the depth of submergence, all of which are points as important as the geographical location of the work. The period in which the teredo accomplishes its work is variable. It may be six weeks or as many years, but rapid work is usually accomplished under the conditions which exist in warm climates.

Impure water and cold weather retard its activity, while pure or warm water expedites the work. Maximum probabilities being more important than minimum possibilities, it is safe to assume that a 6-in. boring may be driven in six weeks, and hence, as the animal attacks all sides, a pile 1 ft. thick may be destroyed in that period.

A young teredo has been found in wood which has been submerged for eight days[a]. Six-inch piles have been destroyed at Aransas Pass in six weeks[b], while other piles in the same locality have lasted for three or four months[c]. "Piles have been rendered useless by a submergence of one hundred days in Mobile Bay." On the Louisville and Nashville Railroad piles 12 x 15 ins. frequently have to be replaced after six months' service[d]. Unpainted spar buoys have a life of about one year in the vicinity of Cape Cod[e]. Piles have been destroyed in the harbor of Galveston in three years[f]. They have lasted twelve years in the Delaware Breakwater Harbor[g].

Reproduction and Development.—Mollusks produce their young by means of eggs. Those of the teredo are spherical in shape and

[a] U. S. Annual Report of Scientific Discovery for 1857.
[b] Report, Chief of Engineers, U. S. A., 1888, pp. 13, 14.
[c] Annual Report, Chief of Engineers, U. S. A., 1879, p. 937.
[d] Montfort, Transactions, Am. Soc. C. E., Vol. xxxi, p. 221.
[e] Report to U. S. Fish Commission, by Capt. Edwards
[f] Report, Chief of Engineers, U. S. A., 1868, p. 512.
[g] Annual Report, Chief of Engineers, U. S. A., 1871, p. 667.

greenish yellow in color. The animal is exceedingly prolific; the eggs of a single specimen being probably numbered by the million. The eggs are first deposited in the gill cavity, and are almost at once fertilized. They are free-swimming at the end of three hours, have a well-developed shell before the end of the day, are very hardy, and all seem to be fertilized and to develop.

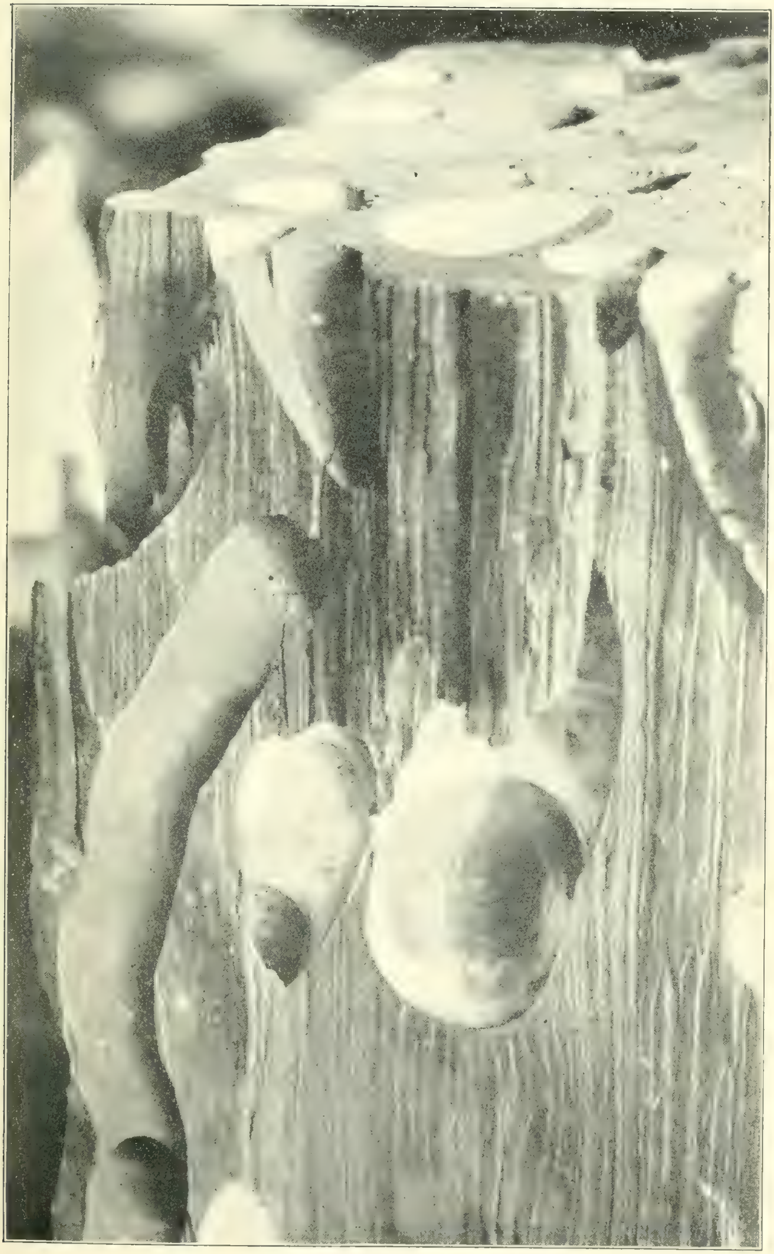
The embryo passes through several interesting stages before it assumes the character and form of the adult. It is first covered by fine hairs or cilia, which enable it to swim. These are soon lost, and the rudiments of a small bivalve shell appear, which is at first heart-shaped and very small, yet large enough to enclose the entire animal. The portion of the body which protrudes from the shell is fringed with cilia. These, again, constitute swimming organs, and the teredo swims actively until a piece of wood is encountered. The shell has now become rounder, and organs of sight and hearing have been developed. The appearance of these organs marks a climax in the life of the young animal, and it begins to elongate. The locomotive cilia disappear, the eyes are lost, and the mature form is gradually assumed. The life of the larvæ is about four weeks, during all of which time they are free swimmers. If the animal has become attached to wood, however, its energies may be expended thereon. The life of a specimen which has not found any wood to attack has not been determined, but is probably quite short.

The results of some observations by Professor Siegerfoos upon the Xylotrya Fimbriata at Beaufort are thus summarized: *

"The free-swimming stage is reached in three hours, and a well-developed shell is formed in a day. We have no direct observations as to the time the ship larva is free-swimming. We may assume, I think, that it is at least a month, or it may be two. Most of its energies are devoted to locomotion during this period, but, after it has attached itself, all of its energies are devoted to forming its burrow and securing its food. Coming in contact with the wood, the larva throws out a single, long byssus thread for attachment and never again leaves its place. The newly attached larva is somewhat less than .25 mm. long. In twelve days, it has attained a length of 3 mm. In sixteen days, 6 mm. In twenty days, 11 mm. In thirty days, 63 mm. In thirty-six days, about 100 mm., when it bears ripe eggs or sperm."

The extreme life limit of the teredo is unknown, but it is thought that under favorable conditions the animal may live for several years.

* Johns Hopkins Circular, 1896.



LIFE SIZE.

In the vicinity of New York the processes of reproduction take place for the most part in May. They are not entirely confined to that month, however, but may extend throughout a greater part of the summer. Reproduction in tropical countries is probably extended throughout the entire year. The animal may develop to a very large size, and may possibly attain maturity in a single season.

The Effect of Climate, Temperature or Water.—The Teredo Navalis thrives best under the influence of heat, but, notwithstanding this fact, it can resist cold to a considerable degree. It is not active when subjected to low temperatures, yet it can endure them. Some species of the teredo have been reported as far north as Eastport, Me., and they exist abundantly under such conditions as obtain at Cape Cod. Destruction is not carried on as continuously or as rapidly in cold climates as in warmer ones, and for this reason maximum results are seen along the South Atlantic and Gulf States and on the Pacific Coast, where the conditions are more favorable, and where reproduction is continued during a longer period.

The syphons of the specimens observed by the author contracted when the water became cold. Only the extremities could be seen when the temperature was about 45 degrees. The syphons expanded as the water became warmer, and were fully extended after several days of continued warm weather. The incurrent tube was stretched out in one instance to a distance of nearly 2 ins. The photograph (Plate XXI) was taken at this period. It is supposed that the work of excavation is not as active when the syphons are withdrawn.

The purity of the water should be considered in connection with the work of the teredo. Some species inhabit pure sea water; some prefer brackish water; others abound in waters that are muddy, while others again live only in waters that are clear and pure.* The teredo is often present in certain waters, yet absent in others nearly adjacent. This is usually due to some difference in the water. The Xylotrya Fimbriata seems to be able to survive the brackish, impure water of the inner New York Harbor, while other species could not live there, though they are present in the nearby outer ocean. The teredo is very active on the North Pacific Coast, yet is absent near the mouth of the Columbia, where the ocean is influenced by the outflow from the river.

* Percival Wright describes a kind of "ship worm" called *Nausitora Dunlopei* found in India, 70 miles from the sea, in perfectly fresh water.

An interesting incident is reported by a reputable firm in New York City. A vessel carrying hard-wood logs was wrecked in the vicinity of the Gulf of Mexico on a sandy beach separating the ocean from a river. The logs were thrown into the ocean, and were afterward beached and conveyed over the sand to the sheltered water of the river, where they remained about six weeks. The wood was rapidly affected as soon as it reached the brackish water of the river, the results being so noticeable that some of the borings were measured, and are said to have averaged 6 ins. in length. The wood which remained in the outer water was not injured.

It is stated that the Russians once built a large dock in the harbor of Sebastopol, and surrounded it with fresh water in the hope that it would be thus protected from the mollusks which infected the harbor; but it was found that the teredo destroyed the wood as rapidly as when it was submerged in salt water. The discrepancy indicated by these incidents may be accounted for by the difference in the species. The teredo which avoids the brackish water at the mouth of the Columbia differs from that which prefers it in the harbor of Sebastopol or in the river near the Gulf of Mexico.

The effect of the condition of the water upon the teredo is interesting. The opinion that the periods of unusual prevalence in Holland were in some way connected with a change in the quality of the water was expressed as early as 1733, and since that time has frequently been endorsed by Dutch engineers. Dr. von Baumhauer, Holland Commissioner to the Centennial Exposition has called attention to the fact that but little rain fell in the years when the teredo was so unusually prevalent, hence the smaller volumes of river water were thought to have permitted larger proportions of salt to reach the coast. This theory is strengthened by the fact that analyses showed a variation in the proportion of salt during dry and rainy seasons.

The Distribution of the Teredo.—The Teredo Navalis has been identified as existing in the United States between Florida and Cape Cod, and in Europe, from Sweden to Sicily. The *Teredo Norvegica* has been found from Cape Cod northward to the coast of Maine. The *Teredo Megotara* has been found in floating pine wood at Newport, R. I., and in cedar buoys, etc., at New Bedford, Mass. It has been found south as far as the coast of South Carolina. The *Teredo Dilatata* occurs from Massachusetts Bay to South Carolina. The *Teredo Thompsoni* has

been found at Cape Cod, Mass. The *Xylophaga Dorsalis* inhabits the waters of the North Atlantic. The *Xylotrya Fimbriata* is found along the Atlantic Coast from Long Island Sound to Florida. It also abounds in the waters of the North Pacific, and is one of the European forms.

Different species of the teredo are notably present in such localities as the Bermudas, Jamaica, New Zealand and Australia. The teredo, as a rule, may be generally found in the Tropics, and is hardly less numerous in many of the northern waters.

Woods Affected by the Teredo.—All varieties of wood commonly used in construction are subject to attack when exposed to the teredo. Immunity is occasionally claimed for some particular wood, but it will generally be found that the claims have been based upon local conditions and are not fully substantiated.

The only woods about which any doubt may be felt are those which contain some gum or bitter essence and those which have a porous structure ; and of these the majority are not well known to American constructors. Some of them undoubtedly possess merit as far as they have been tested, but others may be regarded as open to doubt, and it is an error on the safe side to assume that none are exempt under all conditions. A wood about which less doubt may be felt than any others is the Australian Jarrah. This is a variety of Eucalyptus which is much relied upon in Australia, but it is understood that it has failed in Ceylon, New Zealand and elsewhere. It is very possible, however, that the wood used at the works in question was not the Jarrah.

Karri wood is another form of Eucalyptus, the history of which is much less certain than that of the Jarrah, with which it is often confused. Both varieties are now being extensively introduced into Europe for street-paving purposes. It has often been held that Teak wood is exempt, but the evidence is against it, as Teak wood logs affected by the teredo have been received in New York City within a comparatively recent time.

The following list of partially exempt woods has been compiled by Mr. T. A. Britton from authorities which are said to be reliable:* (Western Australia) Jarrah, Beefwood and Tooart; (Bahama) Stopperwood; (Brazil) Sicupira, Greenheart; (India) Malabar Teak, Sisso,

* Treatise, "Dry Rot in Timber," p. 223.

May-Tobek; (South America) Santa Maria Wood; (Tasmania) Blue Gum; and (West Indies) Lignum Vitæ.

It is not urged that these are entirely exempt, but that they have been exempt for long periods. Very few of them are widely known in construction. It is understood that at Southampton some greenheart piles have failed recently.

A commission, appointed in Holland to investigate this question, decided that—

"Although we do not know with any certainty if among the exotic woods there may not be found those which resist the teredo, we can affirm that hardness is not an obstacle that prevents that mollusk from perforating his galleries."

This conclusion is well borne out by the experience with the Iron Bark tree, the Eucalyptus Leucoxylon, of Australia. This wood has a very great tensile strength, and the crushing strength is said to be "nearly one-fourth that of iron." The wood is certainly very hard, and yet the teredo is by no means repelled by it.

The Eucalyptus Globulus, or Blue Gum, has been successfully introduced into the United States, and is grown in California and Florida. Some hopes have been expressed that this wood might be useful for marine purposes, but they have not been realized. Cedar has repelled the teredo for some little time in the harbor of San Francisco, and it is stated that some species of the Black Mangrove in Jamaica are exempt.

The Osage Orange, or Bodark, has been used to some extent in the Gulf States. It is understood that this wood is relied upon principally because it is hard.

A much greater reliance may be placed upon structure than upon the presence of foreign substances. The teredo desires a compact wood for its abode, and does not like cracks or loose structure. Exogenous trees, palms, for instance, are probably exempt to a greater or less extent. The wood of the latter consists of a mass of thick fibers so independent of one another that brushes can be made by rubbing one end of a stick until the fibers become detached and appear like bristles. Considerable reliance may be placed upon the Cocoa Palm of Mexico, the King Palm of Cuba, and the Cabbage Palm or Palmetto of Florida.

It may be assumed that the conditions of impregnation or structure necessary to repel the teredo do not exist naturally in such woods as

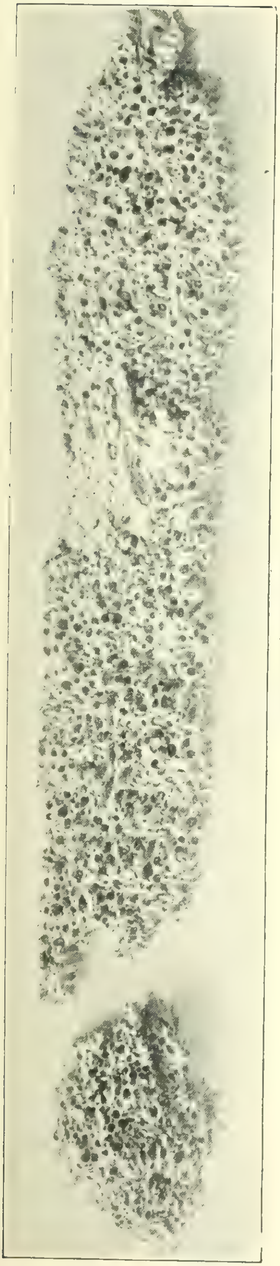


Fig. 1. Life Size.

Fig. 2.—Life Size.

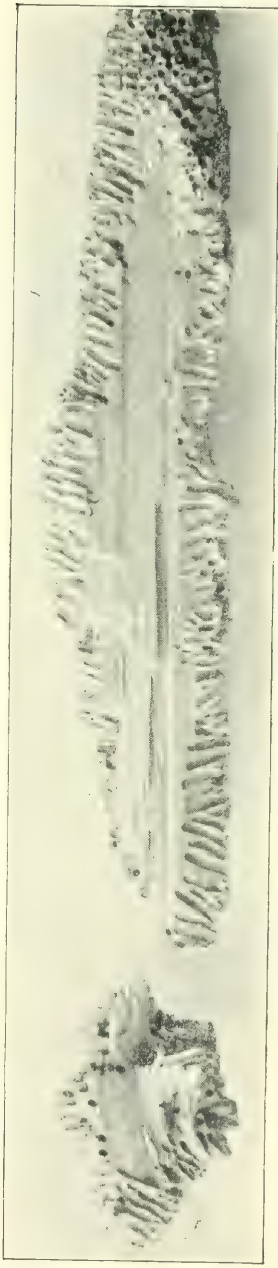


Fig. 3.—Life Size.

are commonly used in engineering works. It may also be assumed, so far as known at present, that partial or complete immunity, as applied to such woods as are in common use, is a question of locality rather than of variety of wood.

The Limnoria Lignorum, White.

This small crustacean has several names, as the Limnoria Terebrans, the Gribble and the Boring Gribble. The limnoria has not been studied for so long a period as the teredo. It was first noticed by Robert Stevenson in 1810, and was examined by Dr. Leach, who one year later pronounced it a new species. It has been investigated since that time by numerous European writers, and, in the United States, it has been studied by Dr. Verrill, of Yale University, and Dr. Sidney I. Smith, of the United States Fish Commission.

The limnoria is gregarious and is found, if at all, in large quantities. It is much smaller than the teredo, but it exists in greater numbers. It has been traced from New York northward to the Bay of Fundy, and large numbers exist in the North Pacific Ocean. It is a very familiar and destructive form of life in Europe. If the destruction accomplished by the limnoria could be estimated it would be found to be surprisingly great.

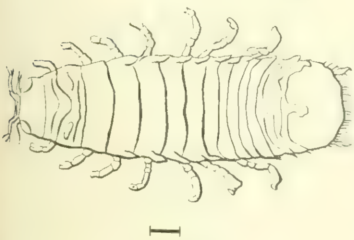


Fig. 3.

Descriptive.—The limnoria (Fig. 3) is about as large as a grain of rice. The body is flat, round at each end, and consists of fourteen segments. The sides are nearly straight and are parallel to one another. To each of the seven segments which follow the head is attached a pair of short, stout legs terminating in claws, the shape of which suggests the small claw of the lobster. The upper surface of the body is covered with small hairs to which more or less dirt usually adheres. The body is grayish in color, and sometimes resembles the color of the wet wood so much that it is difficult to distinguish it. The limnoria can swim, creep backward and forward, as well as jump backward by means of its tail. When touched, it rolls itself into a ball, and in this particular, as well as in general appearance, it resembles the common sow bug.

Vital Processes.—The limnoria differs from the teredo in that it is a vegetarian. The teredo is sustained by infusorial life, but the limnoria devours wood. Its tunnel affords both food and shelter.

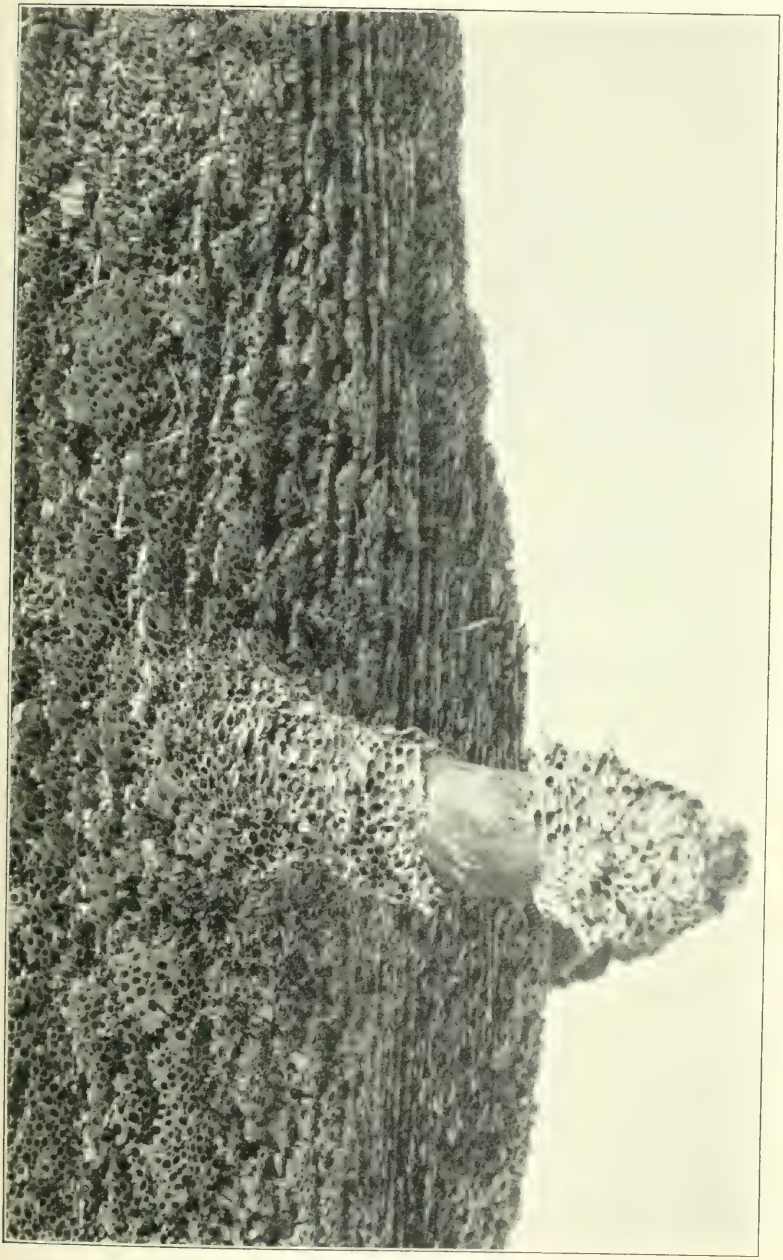
Boring Apparatus.—The limnoria attacks the wood by means of its mandibles or claws. It prefers wet wood and succeeds in making a very clean-cut excavation.

Character of the Excavation.—The work of the limnoria differs from that of the teredo in that it works upon the surface of the wood in such a manner as to be clearly seen, while the work of the teredo is usually concealed until the failure of the wood. The limnoria is similar to the teredo in that its tunnel must communicate directly with the salt water; hence neither of these animals can live in the interior of thick woodwork, such as that of a caisson. The limnoria makes a small, round, parallel-sided tunnel through which it can pass freely back and forth from the sea. The diameter of the entrance of the tunnel is similar to the average diameter. The tunnels are quite short, and are placed very close together (see Plate XXVII). They are so numerous that the wood is rapidly reduced to a series of very thin partitions, which soon decay or are washed away by the waves, thus exposing a fresh surface which is at once attacked. Layer after layer is thus rapidly removed, so that the timber is destroyed in a very few years. The limnoria frequently works in conjunction with the teredo, attacking the exterior while the teredo destroys the interior of the wood, and this combination effects a rapid destruction.

The limnoria attacks both the hard and soft parts of the wood. The hard annual layers have not been avoided in the specimens examined. The limnoria can penetrate knots, but frequently avoids them, so that these hard portions stand out in relief as the timbers waste away. Iron rust is said to cause a somewhat similar effect.

The Size of the Limnoria.—The limnoria is very small, but notwithstanding this fact, it is very destructive. The multitude of these animals compensates for their size. It may be assumed to be from $\frac{1}{6}$ to $\frac{1}{4}$ in. in length, and about $\frac{1}{16}$ in. in diameter. The tunnels are about $\frac{1}{2}$ in. in depth.

The Range or Field of Work.—The wide range observed between the several species of the teredo does not apply to the limnoria. Its work, as observed in the United States, is generally confined to a limited distance above and below the low-water mark. Where the varia-



LIFE SIZE.

tions of the tides are extensive, as in the vicinity of the Bay of Fundy, the range of the limnoria is correspondingly great. The United States Fish Commission states that it has been found, although rarely, as deep as 40 to 60 ft.

The Rapidity of the Work.—The limnoria does not work as rapidly as the teredo. The number of individual workers may be taken as a measure of the work they accomplish. The number of tunnels is more important than their depth. Limnoria are almost invariably found in large numbers and destroy a layer from ¼ in. to 1 in. in thickness in a year, the average yearly destruction being probably ½ in.

Almost all wood used in marine locations is in the form of piles, which are necessarily exposed upon all sides. Their effective diameter may be reduced at the rate of 1 in. for each season, which result, while not equal to that accomplished by the teredo, is sufficient to cause a great loss.

The Effect of Climate, Temperature and Water.—The limnoria is found where the coldness of the climate prohibits the existence of the teredo. It requires pure sea water, and cannot exist in fresh or in impure water, consequently it is not found at the mouths of rivers.

The Distribution of the Limnoria.—The animals are distributed along the American coast from Florida to Nova Scotia. They exist sparingly in Long Island Sound, but are quite numerous upon the coast of Massachusetts, and are very destructive in the Bay of Fundy. They are very active along the North Pacific coast, and are much feared in the vicinity of Puget Sound and the Straits of Fuca. They exist also in abundance upon the coast of Great Britain and in other parts of Europe.

Woods Affected by the Limnoria.—The limnoria seems willing to attack all varieties of wood commonly used by American constructors, but is said to prefer soft woods. It has been known to attack the gutta percha of submarine telegraph cables. It is said that teak wood is free from attack. Plate XXVIII is a life-size photograph of part of a piece of wood from Port Townsend, Wash., showing the work of the limnoria.

Sphæroma Destructor (Richardson).

Attention has recently been called to this hitherto undescribed form of life. This animal is interesting in that it is active in comparatively fresh water. It resembles the limnoria in that it attacks the wood

from without, the interior of the wood being unaffected while the exterior is being destroyed.

The work of these animals was first noticed upon some of the trestles of the Florida East Coast Railway in the vicinity of St. Johns River in Putnam County, Florida. Specimens of the wood were submitted to the Carbolineum Wood Preserving Company, of New York City, and were referred by them to the Smithsonian Institution at Washington, where they were studied by Miss Harriet Richardson.*

The animal somewhat resembles the limnoria in appearance, and is dark brown in color. It works between high and low-water marks. These are not tidal levels, but changes due to the wind assisted by the tides. The water appears to be quite fresh and the water hyacinth, which is not commonly found in salt water, flourishes in the vicinity. The distance to the ocean is about 100 miles.

The diameter of the long-leafed yellow pine pile, from a photograph of which Plate XXIX was prepared, is said to have been reduced from 16 ins. to $7\frac{1}{8}$ ins. in eight years.

The Chelura Terebrans.

This animal was first noticed at Trieste in 1839, and was next found in some piles in the harbor of Kingston. The Irish specimens were described by Professor Allman in 1847.† The chelura was not identified in America until 1875, when two small specimens were discovered by Professor Sidney I. Smith at Wood's Holl, Mass. No others were observed until August, 1879, when Professor Verrill discovered a number of them in some piles at Provincetown, Mass. The chelura unquestionably belongs to the amphipods, and there is apparently but one species of the genus. The C. Pontiac described by Czerniavski in 1868 is identical with the Chelura Terebrans.

Descriptive.—The general appearance of the chelura (Fig. 4) resembles that of the ordinary shrimp, and for this reason is sometimes referred to as the wood shrimp. Its shape differs from that of the limnoria in a very striking degree. The two animals resemble one another only in size. The chelura is a very active little animal, and swims upon its back. It is a jumper, and can project itself to a considerable height when placed upon dry land, and in this respect

* Paper before the Biological Society of Washington, D. C., May 13th, 1897.

† Ann. and Mag. Nat. Hist., xix, 1847, p. 361.

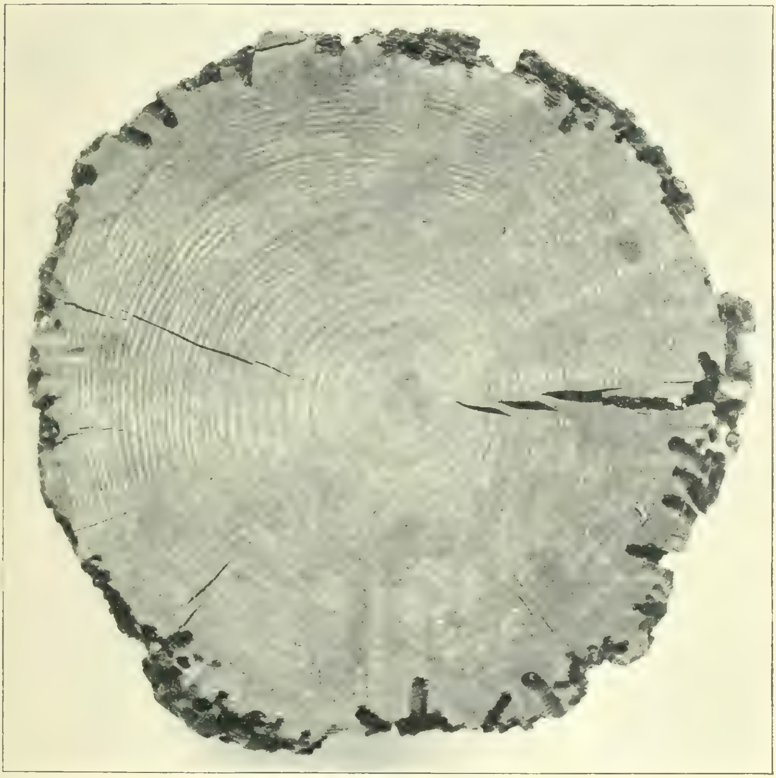


FIG. 1.

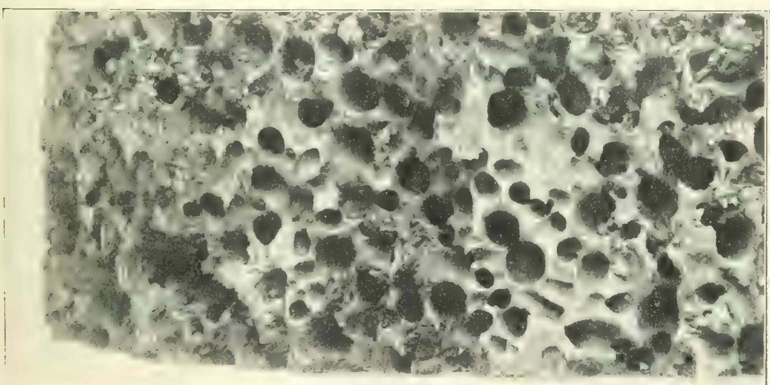


FIG. 2.—LIFE SIZE

resembles the sand hopper. The body is semi-translucent, and is thickly spotted or mottled with pink. The animal is distinguished by three pairs of caudal stylets, the last of which are nearly as long as the body. Those of the females or the young animals are not so long.

Vital Processes.—The chelura resembles the limnoria in that it is a vegetarian, and its burrow affords both residence and food. The fact that the chelura devours wood for sustenance is proved by the minutely divided ligneous matter found in the alimentary organs of dissected animals.

Boring Apparatus.—Professor Allman's original study of the chelura is in part yet regarded as authoritative. He states that the chelura attacks the wood and reduces it to minute fragments by means of a kind of file.

The Character of the Excavation.—Great difficulty has been experienced in obtaining specimens of the work of the chelura, and those obtained are not sufficient to warrant many generalizations. In many particulars the work of the limnoria and of the chelura bear such a close resemblance as to lead to the suspicion that these animals are sometimes confused with one another. The excavations of the chelura are slightly larger than those of the limnoria, but are conducted in much the same manner, as the wood is attacked entirely from without. Numerous punctures are made, and then the weakened layer succumbs to the action of the waves, the surface thus exposed being in turn attacked and the wood destroyed in the same manner. It is stated that the excavations of the chelura are more oblique in their direction than those of the limnoria, and this is certainly true of the specimens observed.

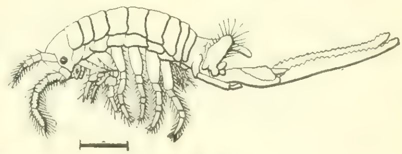


FIG. 4.

The chelura appears to prefer soft wood, and their attacks are made as much as possible in the softer annual rings. The tendency toward an arrangement of perforations in lines is shown in Plate XXX. The work of the chelura differs from that of the limnoria, in that the latter attacks the wood at any available point, while the chelura, on the contrary, prefers the softer portions, and avoids the hard wood around knots. Perforations found in such localities may be assumed to be the work of the limnoria.

The chelura and limnoria are associated with one another in the American localities which have afforded specimens of the former. The perforated wood from these localities shows that the limnoria were in the timber in advance of the chelura. It also shows larger individual chelura confined to the soft rings. These facts indicate that young chelura may at first follow limnoria through the hard rings, but that as they increase in size, they turn toward the soft wood. They obviously attain full size in the larger tunnels. The individual chelura appears to be even more formidable than the individual limnoria.

The Size of the Chelura.—The chelura is somewhat larger than the limnoria. It is said that specimens one-third of an inch in length have been measured.

The Range or Field of Work.—The frequent confusion between these two animals, together with the lack of American data, leaves the question of range unsettled. The specimens found at Provincetown were all taken from wood submerged from 8 to 12 ft. below low-water level.

The Distribution of the Chelura.—The chelura was sought many times along the American coast between New Jersey and Nova Scotia, but was not discovered until 1875. It is yet confined, so far as known, to the two original localities, Wood's Holl and Provincetown, both in Massachusetts, but it is more than possible that the animal has escaped observation, and that it is common on the North Atlantic coast. The unskilful eye would readily confound the chelura with the limnoria, although the two animals belong to distinct divisions of the crustaceans. It is quite possible that some of the damage hitherto ascribed to other animals has been accomplished by the chelura.

The chelura has been reported at many places on the coast of Europe, and is mentioned as existing from South Norway to the Adriatic. Attention has been called to the extent of its range. It is said to be an inhabitant of Australia. In Europe a very great amount of destruction is attributed to this species, and efforts have been made to substantiate these points, but have thus far been unsuccessful. It may be that some European results, attributed to this animal, are deserved by the limnoria, as it is probable that some of the work of the limnoria in America should be attributed to the chelura, and it

PLATE XXX.
PAPERS AM. SOC. C. E.
MAY, 1898.
SNOW ON MARINE WOOD-BORERS.

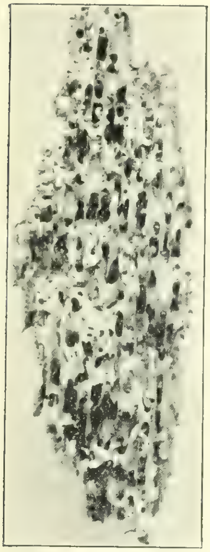


FIG. 2. LIFE SIZE.

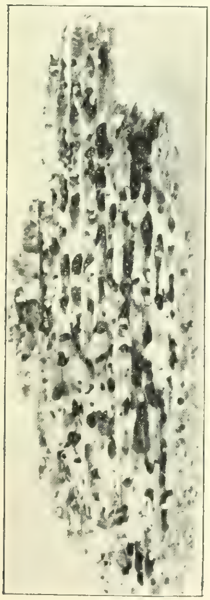


FIG. 1. LIFE SIZE.

FIG. 3. LIFE SIZE.

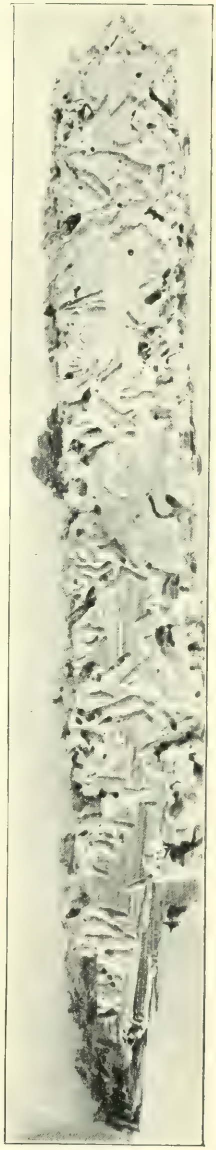


FIG. 4. LIFE SIZE.

is more than probable that the animals are frequently associated. Efforts to discover particular works affected exclusively by this form of life in Europe have not thus far met with success. The chelura has earned a most unenviable reputation in Europe, but it is not known in which places it exists as a specimen and in which as a pest.

Sphæroma Vastator.*

This species was found in the woodwork of a railway bridge on the west coast of the Indian Peninsula. It is said that it honeycombs the wood with cylindrical holes.

The Pholos.

A description of the pholos may suitably accompany any mention of marine wood-borers. This animal does not attack wood, but penetrates the hardest stone, and is therefore interesting because it illustrates the power of boring animals.

The pholos and the teredo are nearly related. The former differs from the latter in that its shell is much larger and in form more closely resembles the long clam. Some species of this animal are much prized by the French as table luxuries. Others are used as food on the North Pacific Coast. The pholos is an inhabitant of many seas; it is plentiful in the English Channel, and is found in many places on the American coast. The borings of the pholos are very instructive from a geological point of view. The so-called Temple of Seraphis, near Naples, affords a prominent illustration of the movement of the earth's crust. This temple has sunk in the water and has then arisen again, the fact of submergence being made clear by the perforations of the pholos. The three principal columns are honeycombed up to a height of about 10 ft., which shows that the ocean once covered the columns to that height.

The method of excavation has already been described in comparison with that of the teredo. The pholos opens its shell so as to brace itself against the sides of its tunnel. The long foot or pestle, which is similar to that of the teredo, emerges and rubs at the surface of the stone. It is assisted by the particles of sand or rock. The cavity is thus enlarged to accommodate the growing animal. Fig. 1, Plate XXXI,

* Described by Mr. Spence Bates in the Ann. of Nat. His., Vol. xvii, 1886, pp. 28–31.

shows one of the numerous species of this animal at work upon a piece of sand-stone. Fig. 2, Plate XXXI, shows a similar animal perforating solid granite.

The series of marine stone-borers is very great, and includes the numerous species of the pholos family, together with other animals not related to them. One of the animals of this class is a powerful enemy of the oyster industry. Another of them destroyed in one year a cargo of marble which had been wrecked in the North Atlantic.

Barnacles.

The barnacle does not perforate wood, but usually attaches itself singly or in clusters to floating or submerged wood, and does not injure it. It is removed from the bottoms of ships because it impedes their progress. Barnacles protect the surfaces they cover. The white blotches on Fig. 1, Plate XXIV, show the places where barnacles were at one time attached. Fig. 3, Plate XXII, shows the form of the barnacle.

Methods of Protection.

A history of the attempts to preserve wood from the attacks of wood-borers would be voluminous. It is only necessary to call attention to those methods which have been attended with more or less success. Most of the attempts in this direction have been made with the idea of protecting wood from the attacks of the teredo. It happens, fortunately, that any method insuring immunity from the teredo secures wood from other wood-borers as well. The methods which have been used may be classified as follows:

Removal During the Breeding Season.—This method may be used to protect such objects as buoys, bathing-houses and row boats. It is only applicable where the breeding season is short as it is in the North.

A Change of Water.—Wooden vessels which have been attacked by the teredo are sometimes hauled into fresh or muddy water. The animals which have gained entrance to the wood are killed by this means. The suggestion has been made that expensive wood-work subjected to the teredo be surrounded by fresh water.

The Use of Selected Woods.—The few varieties of wood for which claims in this particular have been made are not widely known or employed, and it is seldom urged that any are permanently exempt. Repeated attempts have been made to discover some wood upon

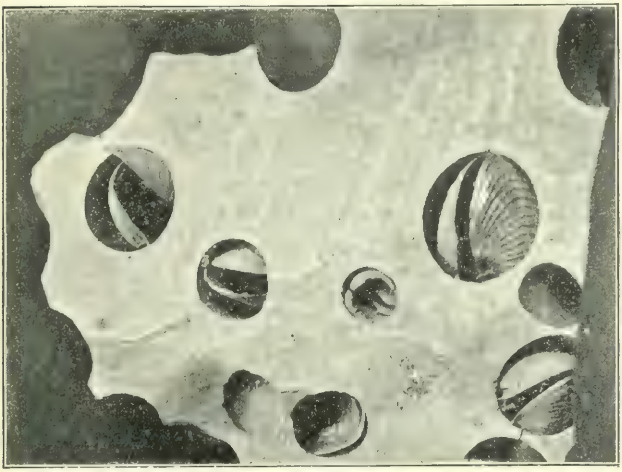


FIG. 1.—LIFE SIZE.

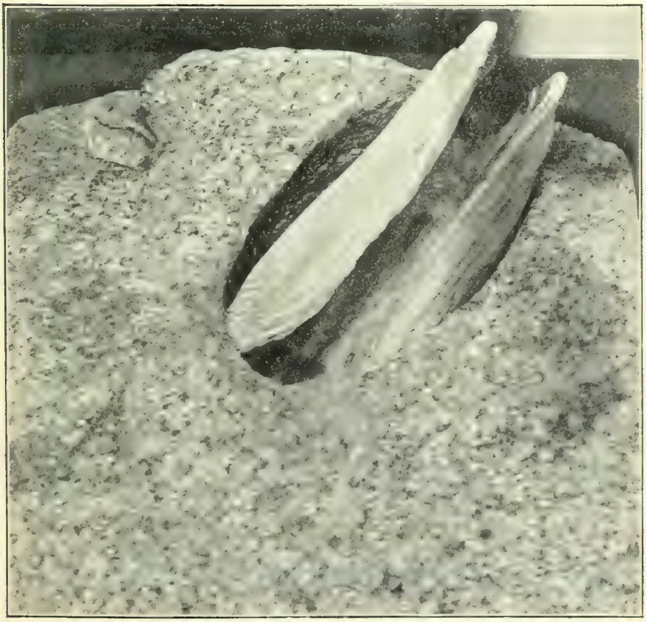


FIG. 2.—LIFE SIZE.

which reliance could be placed, but with meager results. The evidence, thus far, is in favor of the palm and the Australian jarrah wood.

External Coatings.—Many of the protective methods may be grouped under the head of external coatings, one advantage of which is that the treatment may be limited to that portion of the wood which is exposed to attack, while those parts which are below the mud line or above high water need not be considered. This is not the case where internal treatment is used.

(*a*) The bark is sometimes left upon the wood, and, as long as it remains intact, protects it from the teredo. This is explained by the reluctance of the teredo to cross seams. The bark is soon loosened and removed by the waves, however, and the wood is then exposed. It is doubtful whether bark serves as a protection against the limnoria and chelura.

(*b*) Thin plank, joined closely upon the surface of the wood, will preserve it from the teredo during the existence of the plank, but affords no protection from limnoria or chelura, for the plank is soon honeycombed or loosened and the interior woodwork is then exposed to attack.

(*c*) Metallic sheathings, such as copper and zinc, have been successfully used in many places, and the former has proved to be one of the most valuable methods of protection when placed upon piles so that when they are driven home the metal extends below the mud line and up to or above high-water mark. It is used upon the bottoms of wooden vessels and is much superior to zinc, which is quickly acted upon by the salt water. Metallic coatings are expensive, but are very effective, however, in protecting against all forms of marine borers. Surfaces sometimes become coated with barnacles and similar animals which afford further protection to the wood.

(*d*) Teredo or "worm" nails have been extensively used,* and are said to have originated with the Romans. They have short spikes and large, flat and sometimes square heads. They resemble ordinary carpet or upholsterers' tacks, and are driven close together. According to the specifications adopted by the Dutch Waterstaat, the nails must be well forged and not brittle. The diameter of the head must be 3 cm. and the length of the tack 4 cm. One kgm. is to contain

* The Dutch use them to a height of one-half tide.

from thirty to thirty-four nails. An interesting series of experiments with teredo nails has been conducted by the New York Department of Docks.* Teredo nails are apt to rust and thus cause failure. It was once held that the iron rust impregnation assisted in repelling the teredo, but this appears to have more foundation of truth as regards the limnoria.

(*e*) Paints, verdigris, paraffine, tar, asphalt and other mixtures have been used as protectives, but it is usually difficult to retain such coatings in position. Mixtures which withstand the softening effect of sea water are likely to be removed by erosion, and surfaces should be inspected at least once a year.

(*f*) Attempts have been made to combine paint mixtures with some fabric such as burlap or wire netting. Asphalt and net have not proved successful on the Pacific coast,† but a combination of paraffine paint and burlap used there has attracted attention. After removing the bark the surface of the pile is covered with a prepared compound, some of the ingredients of which are paraffine, powdered limestone and kaolin. The pile is then wrapped in jute burlap, and another application of the compound is made. Wooden battens are then nailed along the surface, which receives a final coat of the paint. Piles thus protected have been in use for ten years on the Pacific Coast by the California State Board of Harbor Commissioners, by the Northern Pacific, Great Northern and other railways, and are said to have been successful. The coating protects the piles from the teredo, limnoria and similar animals, but its duration is not known.†

(*g*) Piles are sometimes covered with Portland cement mortar. The bark is first removed and the wood cleared of knots and similar projections. The pile is then driven to its final position. The mortar is applied in several ways. A jacket of ordinary sewer pipes extending from the mud line to high-water mark is sometimes placed around the pile, and the space thus enclosed filled with hydraulic cement. Piles thus protected have been put in position and observed by the California State Harbor Board. The coatings were soon found to have cracked, probably because they were too stiff. An iron shell or mould made in two pieces, bolted together tightly around the pile has sometimes been used. The intervening space is filled with cement, and the

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* *Transactions*, Vol. xxxi, p. 235.

† *Engineering News*, February 8th, 1894.

mould removed as soon as the cement has become hard. The Louisville and Nashville Railroad treated four thousand piles in this way, at an average cost of $1.25 per foot of length.* The cost of repairs for the first seven years was comparatively small. The concrete became coated with oysters and barnacles and was thus further strengthened. The advantage of such a treatment is that it can be applied after the piles have been driven. Teredos or limnoria may unexpectedly attack the wood, and any specimens which have gained entrance can be killed and others repelled by this method. The cost is not as great as might appear, since the entire length of the pile is not covered.

(*h*) The use of sand has been found to be both effective and low priced. Cylinders of earthenware pipes joined together by a special cement, are lowered over the pile and pushed into the bottom. The space between the cylinder and the pile is filled with sand. Any fracture or leakage is made evident at the top and can at once be made good. This method was suggested by the Louisville and Nashville Railroad, and is considered to be an improvement on their former method of protection by means of cement, while the cost, about 70 cents per foot, is much lower. The method is said to insure greater elasticity and better protection at the bottom. Piles treated in this way on the New Orleans and Mobile Division are apparently as sound as when driven, twenty years ago. In some cases their tops were not covered with pitch when they were sawed off, and the heart wood of a few of these specimens has decayed. The outer sapwood still remains sound.†

(*i*) External protection is sometimes afforded naturally. The surface of the wood may become covered with barnacles, mussels, oysters or similar animals, and is thereby protected from attack. Sea thorns sometimes multiply to such an extent that the entire surface is covered by their disks, which afford a very effective protection.

A Dutch commission, after six consecutive years of investigation, reported that coatings applied to the surface of wood seemed insufficient; that such coatings are likely to be injured by mechanical means; that chemical changes are to be looked for; and that it is difficult to obtain a covering which will continue in close contact with the wood.

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* *Transactions*, Vol. xxxi, p. 225.

† *Transactions*, Vol. xxxi, p. 221.

The subject of external coatings may be thus summarized : Protection may be afforded as long as the coating remains intact, but this is difficult to accomplish.

Internal Treatment.—Many substances, such as water-glass, the salts of mercury and of iron, have been suggested as substitutes for coal-tar creosote, but none of these can compare with it, and therefore deserve no further notice.

Creosote supplies the best means for repelling the attacks of the teredo, limnoria and other sea animals, and also the termite and other land wood-borers. The subject of creosoting divides itself into three parts: the creosote, the method and the wood.

Creosote is a substance which is contained in the second distillation of coal tar. The first distillation consists of light oils, the second, creosote, and the third, pitch. Tars differ greatly in their chemical constituents, and in their products of distillation. The word cresote, therefore, has not an absolutely exact definition. The substance has no chemical symbol, as it applies to a fluid, the constituents of which constantly differ. It is essential that creosote should be heavier than water, as light creosotes have never been satisfactory, and most of the failures attributed to creosote have really been due to the use of such oils.

Creosote is expected to act in two ways. It introduces antiseptics into the wood; it also fills the pores with thick, gummy insoluble oils and naphthaline. Therefore, a second distillate of coal tar, which contains antiseptics and gummy substances in sufficient quantity and of satisfactory quality, should be selected. It should contain over 40% of naphthaline, and as little pitch as possible. It may contain as much carbolic acid as is likely to be present in this distillate, which will not be over 4 or 5 per cent. No substances likely to accompany the minimum of 40% of naphthaline will be injurious, because many of them may be regarded simply as vehicles. Heavy oil of creosote is heavier than water, and is sufficiently insoluble to remain in the wood for a long time. Creosote weighs from 8 to 9 lbs. to the gallon. The United States cannot meet the demand for dead oil of coal tar, and, therefore, a large quantity is derived from England. The so-called "London Oil" is very thick and heavy. It is thought to be one of the best grades of creosote for marine work.

The method by which creosote is introduced into the wood is most important, but any method which will insure a thorough impregna-

tion will be satisfactory. The wood is first heated in a vacuum to remove the moisture. The heat is so manipulated as to vaporize the sap and coagulate the albumens of the wood. Heated creosote is then introduced, and the condensation of the vapor in the wood causes a vacuum which, assisted by pressure, draws in the creosote. A gauge outside of the tank indicates the subsidence of the creosote as it passes into the wood. The process is stopped as soon as the specified quantity of creosote, usually from 10 to 16 lbs. per cubic foot of wood, has been forced in.

The selection of the wood which is to receive the creosote is important. Some woods are more porous than others, and one which will permit the free entrance of creosote is better than one which is hard or otherwise durable. It should be of such a nature that it will protect the creosote after impregnation. Creosote has occasionally failed because it has not been used in connection with wood of the proper quality. The Georgia pine and the Loblolly pine are the best for this purpose. Green woods are sometimes preferred to those which have been seasoned, because the condensation of the vaporized sap assists in more thoroughly impregnating the wood.

Many cases of failure are recorded against creosoted wood. Other cases are on record of woods which have resisted at first, but have succumbed after several years of exposure. In all these cases it may be assumed that the creosote was at fault; that the work was not thoroughly done, or that the wood was of a kind which resisted the ordinary treatment. A well-selected wood, thoroughly impregnated with good coal tar creosote, will resist the teredo, the limnoria, and probably all other forms of life for many years.

Substitution.—Substitution can hardly be classified as a method of preservation, but should be mentioned in connection with this subject, because the use of iron in ship-building is constantly increasing. Were this not so, marine wood-borers would require much more attention than they receive at present. Iron piles are used to some extent, and the use of iron work in marine construction may be safely said to be on the increase. There are many marine works, however, in which iron can hardly be used as a substitute for wood.

The author has received notable assistance from Professor Verrill, of Yale University; Professor Packard, of Brown University; Professor C. O. Siegerfoos, of Johns Hopkins University; the National Museum

at Washington, and General John M. Wilson, Chief Engineer of the United States Army. The New York Museum of Natural History has permitted the author to photograph some of its specimens. The United States Fish Commissioner has contributed much data, and prepared drawings of the teredo, the limnoria, and the chelura. The New York Aquarium, Colonel William Ludlow, M. Am. Soc. C. E., Francis Collingwood, M. Am. Soc. C. E., and many others have been of assistance.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

By Charles Warren Hunt, M. Am. Soc. C. E.

Cloth, 6x9 Inches.

Printed by order of the Board of Direction of the American Society of Civil Engineers, to be sold only on subscription. The proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

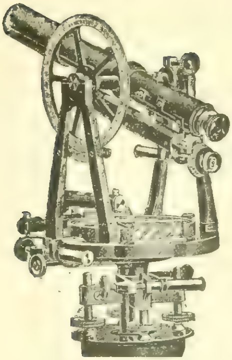
In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

This action has been taken in the belief that many members will welcome the opportunity of contributing something to the building fund.

There are a few copies still on hand of the first lot bound and these can be obtained by those who so desire at $10 per copy.

Orders should be sent to the Secretary.

The book begins with a brief statement of the first movement to form a National Society of American Engineers in 1839. The organization of the American Society of Civil Engineers and Architects in 1852 is then described, a list of its promoters and charter members given, and the work accomplished in its first two years of life sketched. The reorganization of the Association in 1867 and the important events in its career from that date to 1873, when the first publication was issued, are then given in chronological order. Succeeding chapters are under the following heads: Locations Occupied by the Society, Library, International Exhibitions, Publications, Badge, Constitutional Changes and Work Accomplished. Under the head of "Comparative Growth of National Engineering Societies" short sketches of the Institution of Civil Engineers and the Société des Ingénieurs Civils are given. The illustrations consist of 35 half-tone portraits of past officers of the Society and one diagram, all handsomely printed on heavy paper.



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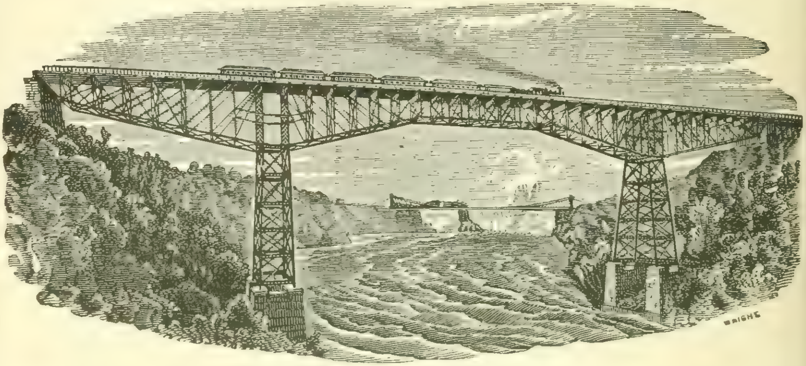

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

June, 1898

PROCEEDINGS = VOL. XXIV—NO. 6

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PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publication.

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SOCIETY AFFAIRS.

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ILLUSTRATIONS.

The prices of publications are as follows: Proceedings, $6 per annum; Transactions, $10 per annum. Postage will be added when they are sent to foreign countries.

American Society of Civil Engineers.

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The House of the Society is open from 9 to 22 o'clock every day, except on Sundays, when the hours are from 14 to 19 o'clock.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - 2236–38th Street.
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Vol. XXIV. JUNE, 1898. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

June 1st, 1898.—The meeting was called to order at 20.30 o'clock, James Owen, Director, in the chair; Charles Warren Hunt, Secretary, and present also 109 members and 22 guests.

The minutes of the meetings of May 4th and 18th, 1898, were approved as printed in *Proceedings* for May, 1898.

Robert B. Stanton, M. Am. Soc. C. E., delivered an address entitled "The Cliff Dwellers of the Far Southwest; Their Homes, Their Agricultural and Engineering Works and Their Military Knowledge and Art," which was illustrated by the stereopticon.

Ballots were canvassed and the following candidates declared elected:

As Members.

William Edgar Baker, Chicago, Ill.
William Robert Browne, Pittsburg, Pa.
Standish Barry Burton, Monterey, Mexico.
Lyman Edgar Cooley, Chicago, Ill.
Robert Augustus Cummings, Philadelphia, Pa.
Josephus Conn Guild, Chattanooga, Tenn.
Edward Henry Hurry, Bethlehem, Pa.
Francis Robert Johnson, Capetown, Cape Colony, S. A.
Walter Camp Parmley, Cleveland, O.
Alfred Howard Renshaw, Troy, N. Y.
Charles Hopkins Van Orden, Catskill, N. Y.

As Associate Members.

Frederick Anderson Burdett, New York City.
Frederick Edward Ferris, Jersey City, N. J.
Robert Dickson Alison Frew, Strathfield, N. S. W., Australia.
Robert Maxson Greene, New York City.
Eugene McLean Long, Pittsburg, Pa.
Enos Larkin Shaw, Green Bay, Wis.
Edward Burnham Stearns, Boston, Mass.
Charles Morton Strahan, New York City.
Elton David Walker, Schenectady, N. Y.

The Secretary announced the election, by the Board of Direction, on May 31st, 1898, of the following candidates:

As Associate.

John Monks, Jr., New York City.

As Juniors.

John Walker Barriger, Jr., Tyler, Tex.
James C. Hain, Chicago, Ill.
Clarence William Hubbell, Detroit, Mich.
Charles Joseph Tilden, Allegheny, Pa.

The Secretary announced the death of Charles Edward Emery, elected Member, May 6th, 1874; died, June 1st, 1898.

Adjourned.

June 15th, 1898.—The meeting was called to order at 20.30 o'clock, President Alphonse Fteley in the chair; Charles Warren Hunt, Secretary, and present, also, 76 members and 23 visitors.

A paper by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., entitled, "Origin of the Gulf Stream and Circulation of the Waters in the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," was presented by the Secretary, who also read correspondence on the subject from A. F. Wrotnowski, Thos. L. Raymond, Alexander E. Kastl, George Y. Wisner and H. C. Ripley. The paper was discussed orally by Messrs. L. M. Haupt, Thomas D. Pitts and Brewster Cameron.

The Secretary announced the programme for the Thirtieth Annual Convention, to be held in Detroit, Mich., July 26th–29th. 1898.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 31st, 1898.—President Fteley in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Clarke, Deyo, Morison, Owen, Parsons, See and Thomson.

Reports of the Board of Direction to the Society on the proposed appointment of Special Committees to report on "Rail Joints for Standard Steam Railroads," and on "Paints Used for Structural Work in Engineering" were adopted.

One reconsideration ballot was canvassed.

Applications were considered and other routine business transacted.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society will be open every day hereafter from 9 to 22 o'clock, except on Sundays, when the hours will be from 14 to 19 o'clock.

JUNE NUMBER OF PROCEEDINGS.

Owing to the change in the time of holding the Annual Convention of 1898, the Board of Direction has decided that the present Number of *Proceedings* shall be issued, and shall take the place of the number for August, 1898.

THIRTIETH ANNUAL CONVENTION.

A circular, giving the general programme of the Thirtieth Annual Convention, to be held in Detroit, Mich., July 26th–29th, 1898, has already been mailed to each member of the Society.

It is specially requested that members intending to go to the Convention should notify the Secretary at as early a date as possible.

MEETINGS.

Wednesday, September 7th, 1898.—The first regular meeting of the season 1898–99 will be held at the Society House at 20.30 o'clock.

DISCUSSIONS.

Discussion on the paper by R. S. Buck, M. Am. Soc. C. E., entitled, "The Niagara Railway Arch," which was presented at the meeting of May 18th, 1898, will be closed July 1st, 1898.

Discussion on the paper by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., entitled, "Origin of the Gulf Stream and Circulation of the Waters of the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," which was presented at the meeting of June 15th, 1898, will be closed August 1st, 1898.

LIST OF MEMBERS.

ADDITIONS.

MEMBERS.			Date of Membership.
CUMMINGS, ROBERT AUGUSTUS	714 Girard Bldg., Philadelphia, Pa		June 1, 1898
GRESHAM, ROBERT HALL	Asst. City Eng. of San Antonio, Tex. (Res. 209 Elm St., San Antonio, Tex)		May 4, 1898
GUILD, JOSEPHUS CONN	Guild & White, Chattanooga, Tenn		June 1, 1898
JOHNSON, ARCHIBALD	Local Charge U. S. Reservoirs in Minnesota—The Portland, St. Paul, Minn		April 6, 1898
O'MELVENY, JOHN CHARLES	Chf. Eng., Oregon Short Line R. R. Co., Salt Lake City, Utah		May 4, 1898
PARMLEY, WALTER CAMP	19 Burt St., Cleveland, Ohio	Assoc. M. M.	April 1, 1896 June 1, 1898
RENSHAW, ALFRED HOWARD	Prest., Trojan Car Coupler Co., Troy, N. Y		June 1, 1898
STRACHAN, JOSEPH	Asst. to Eng. Water Supply, Brooklyn, N. Y. (Res. 352 Putnam Ave., B'klyn, N. Y.)	Assoc. M. M.	May 4, 1892 May 4, 1898
TAYS, EUGENE AUGUSTUS HOFFMAN	Mining Eng. to Anglo-Mexican Mining Co., Ltd., of San José de Gracia, San José de Gracia, Estado de Sinaloa, Mexico		April 6, 1898
WILLIAMS, DAVID	Asst. Eng., St. Johnsbury and Lake Champlain R. R., St. Johnsbury, Vt.		May 4, 1898

ASSOCIATE MEMBERS.

ALLEN, FRANK WILLIAM	845 Eighth Ave., New York City.	Jun. Assoc. M.	Nov. 3, 1896 May 4, 1898
BALDWIN, ERNEST HOWARD	Asst. Eng., Reservoir Dept., Metropolitan Water Works, 23 Prospect St., Clinton, Mass.		May 4, 1898

		Date of Membership.
BROOK-FOX, EVELYN	Nawada Gya District, Bengal, India	Feb. 2, 1898
CHESTER, CHARLIE ELLSWORTH	Paola, Kan	May 4, 1898
COOK, JOHN HENRY	Care of Passaic Water Co., 107 Washington St., Paterson, N. J	May 4, 1898
GREENE, ROBERT MAXSON	Care of Milliken Bros., 39 and 41 Cortlandt St., New York City	June 1, 1898
LONG, EUGENE McLEAN	20 Craig St., Pittsburg, Pa.	June 1, 1898
SCHNEIDER, ANTON	Asst. Engr., Oregon Short Line R. R. Co., Salt Lake City, Utah	May 4, 1898
SIRRINE, JOSEPH EMERY	Care of Indian Head Mills of Alabama, Cordova, Ala	Jun. June 20, 1893 Assoc. M. May 4, 1898
WALKER, ELTON DAVID	Asst. Prof., C. E., Union College, Schenectady, N. Y.	Jun. Oct. 4, 1892 Assoc. M. June 1, 1898
WINFREE, PEYTON BROWN	City Engineer, Bradford, Pa.	Jun. Oct. 2, 1894 Assoc. M. May 4, 1898

ASSOCIATES.

HARTRANFT, WILLIAM GARRIGUES	Vice-Prest., Commercial Wood & Cement Co., 305 Girard Bldg., Philadelphia, Pa. (Res. Camden, N. J.)	May 3, 1898
HUSTON, JAMES ARCHIBALD	Prest. and Gen. Mgr., Toledo Bridge Co., Toledo, Ohio	Aug. 31, 1897
MONKS, JR., JOHN	John Monks' Son, 130 Water St., New York City	May 31, 1898

JUNIORS.

HARRINGTON, JOHN LYLE	Asst. Supt., Structural Dept., Cambria Iron Works, Johnstown, Pa.	Aug. 31, 1897
McFETRIDGE, WILLIAM SUTTON	Asst. Engr., Pittsburg, Bessemer and Lake Erie R. R., Greenville, Pa.	May 3, 1898

CHANGES AND CORRECTIONS.

MEMBERS.

Adgate, George....................68 McLaren St., Ottawa, Ontario, Canada.
Barlow, John Quincy.............Kemmerer, Wyo.
Blackwell, Charles...............4400 St. Catherine St., Montreal, Canada.
Bowman, Austin Lord.............Room 1301, 71 Broadway, New York City.
Cheever, Albert Stafford........Secretary to Pres't, Fitchburg R. R., Boston, Mass.
Covode, James Henry.............Engineers' Club, 374 Fifth Ave., New York City.
Dunham, Herbert Franklin.......2 Wall St., New York City.
Filley, Hiel Hamilton...........2912 Charlotte St., Kansas City, Mo.
Flagg, Josiah Foster.............Flagstaff, Arizona.
Francis, Henry Newton...........Sup't, Narragansett Imp. Co., 122 Public St., Providence, R. I.
Gardiner, Edmund Le Breton.....Passaic, N. J.
Harrison, Charles Lewis..........Schuylersville, N. Y.
Jaques, William Henry...........Pres't, Holland Torpedo Boat Co., 141 Broadway, New York City.
Johnston, Andrew Langstaff......Cons. Eng., Meadville Traction Co., Meadville, Pa.
Lea, Samuel Hill.................Asst. Eng., Allegheny Western Ry., Adrian, Pa.
Lowrie, Harvey Childs...........The Denver Club, Denver, Colo.
MacFarlane, James...............Box 484, Butte, Mont.
Mosman, Alonzo Tyler............Asst., U. S. Coast and Geodetic Survey, Santa Barbara, Cal.
Noyes, Ellis Bradford...........200 Dinwiddie St., Portsmouth, Va.
Pearson, Fred Stark.............Room 840, Cable Building, New York City.
Phillips, Joseph Leslie...........Compania de Inguaran, Ortega 28, City of Mexico, Mexico.
Rutherford, Francis Morris......202 West 78th St., New York City.
Scott, Ernest Kay...............7026 Butler St., Chicago, Ill.
Sengoku, Mitsugu...............Kiushiu Railway, Moji Fukuokaken, Japan.
Sloan, Robert Imlay..............Park Ave., Elberon, N. J.
Stern, Eugene Washington........56 West 23d St., New York City.
Taylor, Norton Longstreth......City Engineer, Tacoma, Wash.
Whitford, Oscar F................79 Woodlawn Ave., Buffalo, N. Y.
Wilgus, William John............Engr., M. of W., N. Y. C. & H. R. R. R., 48 Grand Central Depot, New York City.
Wrotnowski, Arthur Francis.....Guaymas, Sonora, Mexico.
Yates, Preston King.............Canajoharie, N. Y.

ASSOCIATE MEMBERS.

Barber, William Davis...........48 Evergreen Ave., Chicago, Ill.
Barney, Samuel Eben............Grove Beach, Conn.
Christian, George Lyon..........East 235th St., Woodlawn, New York City.

Creuzbaur, Robert Walter.......Room 55, 280 Broadway, New York City.
Diebitsch, Emil..................18 Oak St., Buffalo, N. Y.
Fisher, Elstner..................Gen. Supt. and Chf. Engr., T. H. & B. Ry., Hamilton, Ontario, Canada.
French, James Benton............Bridge Eng'r, C. & O. Ry. Co., Richmond, Va.
Hardy, Harry....................St. Moritz, 48 Eversfield Place, Hastings, Sussex, England.
Meem, James Cowan..............193 Warren St., Brooklyn, N. Y.
Rhys-Roberts, Edmund Alyth.....Care of Dominion Bridge Co., Ltd., Montreal, Canada.

JUNIORS.

Burdett, Frederick Anderson....334 Madison St., Brooklyn, N. Y.
Coulson, Benjamin Le Fevre......Hodgenville, Ky.
Ellis, George Ezra..............Room 18, New York Central Station, Albany, N. Y.
Fish, John Charles Lounsbery....Florence, Erie Co., Ohio.
Ford, William Hayden............Care of U. S. Consul, Rosario, Argentine Republic, S. A.
Horton, Sandford.................Avenida Balderas, No. 631, City of Mexico, Mexico.
Horton, Thomas Osborn..........New Amsterdam Gas Company, 340 Third Ave., New York City.
Houston, Gavin Nelson...........Nyack, N. Y.
Monsarrat, Nicholas Daubeney....Care of Peoria and Pekin Traction Co., Niagara Bldg., Peoria, Ill.
Phillips, Howard Crathorne......Amarillo, Tex.
Prince, Arthur Dickson..........26 West 50th St., New York City.
Rittenhouse, Harvey..............Fairview Village, Montgomery Co., Pa.
Rosenthal, Albert...............Care of The United Gas Improvement Co., 1315 Filbert St., Philadelphia, Pa.
Rowell, George Freeman.........75 Hicks St., Brooklyn, N. Y.
Russell, Richard Lord...........Care of Nassau Electric R. R., Third Ave. and 1st St., Brooklyn, N. Y.
Shaw, Enos Larkin...............Care of Pennsylvania Steel Co., Steelton, Pa.
Swaab, Solomon Mark ....Care of Bureau of Surveys, City Hall, Philadelphia, Pa.
Swindells, Joseph Springer.......Care of Samuel Swindells, 316 Throop Ave., Brooklyn, N. Y.
Vanderlip, Henry Eleaser........3960 Drexel Boulevard, Chicago, Ill.

FELLOW.

Clement, Victor M...............Care of W. K. Millar, 24 Austin Friars, London, E. C., England.

DEATH.

Emery, Charles Edward..........Elected M. May 6, 1874; died June 1, 189[illegible]

1898. JUNE. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS:

DREDGES AND DREDGING ON THE MISSISSIPPI RIVER.

BY J. A. OCKERSON, M. AM. SOC. C. E.

TO BE PRESENTED AT THE ANNUAL CONVENTION, JULY, 1898.

The Mississippi River is utilized for purposes of navigation from the Gulf of Mexico to the Falls of St. Anthony, a distance of about 2 000 miles.

In this great length, the character of the stream goes through various changes, through the influx of tributaries and other causes.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

In order to get a clear comprehension of the controlling physical conditions in different parts of the stream, it is best to divide it into four distinct reaches or sections.

First.—That portion extending from the Falls of St. Anthony to the mouth of the Missouri, a distance of 712 miles. In this reach the banks are low, and the oscillation between high and low water rarely exceeds 25 ft. In the upper half of this reach the river is divided into a great many sloughs, which serve as high-water channels, but are often nearly or quite dry at low water. The water carries but little sediment; bank erosion is comparatively slight; for 21 miles it flows through a lake of slack water 30 ft. deep; the flow in two places is interrupted by rapids where the bed of the stream is solid rock; in the upper portion, the navigable depth at low water sometimes gets down to $2\frac{1}{2}$ ft., and navigation is usually suspended during the winter season for a period of four months or more in consequence of the river being frozen. The low-water slope averages about 0.5 ft. per mile. The low-water discharge about 25 000 cu. ft. per second, and the high-water discharge about 350 000 cu. ft. per second. High water generally comes in May and June, and the low-water season usually begins about the first of September and lasts until navigation is closed by ice.

One of the rapids spoken of, near Rock Island, Ill., 1 575 miles above the mouth, has been improved by the removal of rock and the concentration of the volume by dikes and dams. The other, above Keokuk, Ia., 1 445 miles above the mouth, is surmounted by means of a canal, with 8 miles of slack-water navigation and three locks, with a total lift of 18 ft.

Second.—The second reach extends from the mouth of the Missouri to the head of the alluvial basin of the lower river, or practically to the mouth of the Ohio, a distance of 210 miles. This reach is the first to take up the enormous load of sediment put upon it by the Missouri River. Here permeable dikes are at their best, and immense deposits are easily induced where channel contraction is desirable to increase the depth. The banks are somewhat higher than those of the first reach, and the effects of bank erosion are more noticeable. The extreme oscillation between high and low water near the upper portion of this reach is some 36 ft.; the low-water slope averages 0.6 ft. per mile; the low-water discharge is about 45 000 cu. ft. per second, and the high-water discharge about 850 000 cu. ft. per second. At low water the

navigable depths on the bars often reach as low as 4 ft. Overflows are not very frequent, as a conjunction of the floods of the upper Mississippi and Missouri Rivers is necessary to produce an overflow stage. The high-water stages usually occur in May and June, and the low-water season begins early in September and continues into the winter months.

Sand bars are numerous, and the crossings are consequently frequent, and their locations are constantly shifting. The river washes the rocky bluffs on one side or the other a greater part of the distance, and at Gray's Point, 1 100 miles above the mouth, it flows through a rocky gorge for a distance of 7 miles. At the lower end the normal conditions are often complicated by back-water from floods in the Ohio, which causes the sediment to deposit, and this is a prolific source of annoyance to navigation.

This reach is sometimes frozen over for a month or so during the winter, but is more often free from ice throughout the year. The whole reach partakes much of the character of the Missouri River.

Third.—This reach extends from the mouth of the Ohio to the mouth of the Red River, a distance of 750 miles. Here the Ohio, reinforced by the Tennessee and Cumberland Rivers, comes in and controls the flood conditions of the lower river. The Missouri River no longer dominates, except in its mud; the bed of the stream is now through the deposits which it has built up and torn down repeatedly; at a few points the river is held in check where it strikes the bluffs, but for the most part the banks yield readily to the eroding power of the current. At one place a straight reach becomes excessively wide, as near New Madrid, by the river encroaching first on one bank and then on the other; at another place it becomes exceedingly crooked, as near Greenville and above, by the continued erosion of the concave bank and the building out of the point opposite the bend. The caving reaches enormous proportions, the maximum lying in about the middle third of the reach; the oscillation between high and low stages reaches over 53 ft. The banks are high; overflows are frequent; the sand bars are very large in extent; islands and towheads are numerous, and the width of the river here reaches the maximum; the bars which obstruct low-water navigation are not very numerous, and the depth rarely gets below 5 ft. on these bars, while by far the greater part of the reach has water of ample depth to satisfy all demands of naviga-

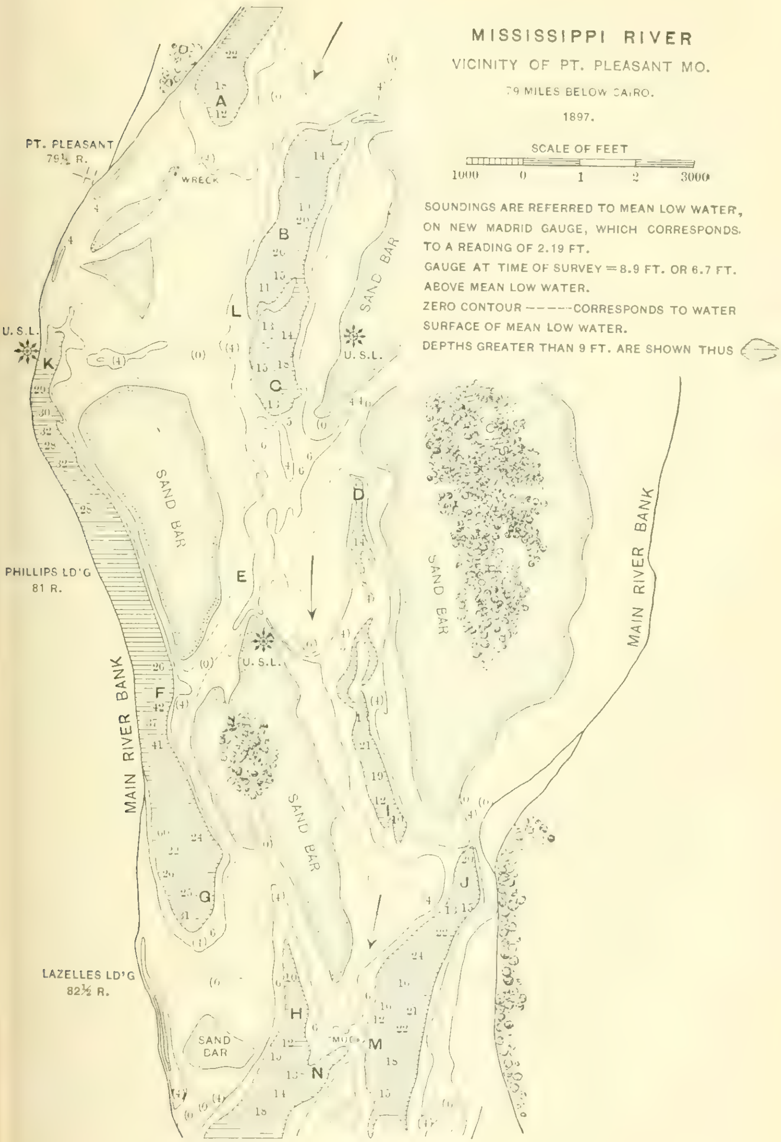
tion; the river is rarely obstructed even by floating ice; low-water conditions which interfere with navigation rarely exceed four months in duration, and now and then entire years pass without any serious interruption to navigation; the low-water slope averages about 0.35 ft. per mile; the floods usually come in February or March, and the low-water conditions from September to December; many of the bends have depths of over 100 ft., and the discharge varies from 65 000 cu. ft. per second at extreme low water to 2 000 000 cu. ft. per second at extreme high water.

The work of maintaining navigable channels through the crests of the bars by means of dredging has, so far, been largely confined to this reach.

Fourth.—The fourth reach extends from the mouth of the Red River to the Gulf of Mexico, a distance of 310 miles. In this portion of the river the channel is narrow and deep, the banks tolerably stable, and sand bars as obstructions to navigation are almost unknown. So far as navigation is concerned, this reach requires neither contraction works nor dredging. Nature has built a channel which man vainly tries to imitate in the reaches lying above. Only two islands exist in this reach, and the last gravel bar is near the upper end of the reach. The upper limit at low water is only 3 ft. above mean Gulf level, and the tidal effect is often observed over the whole length of the reach. The first practicable outlet, for the waters coming down, is through the Atchafalaya at the head of this reach. The extreme oscillation between high and low water at the upper end of the reach is 50.4 ft., and this, of course, tapers down to zero at the Gulf. The river in this reach carries the burdens of all the tributaries coming from a drainage area of 1 250 000 square miles. The destructive floods invariably come from the Ohio basin, augmented at times by the tributaries below the mouth of the Ohio River.

The characteristics of the reaches described are pronounced, and they differ so widely as to make the improvement of each a problem by itself. The last-named reach should perhaps have terminated at the Head of the Passes instead of at the Gulf, 12 miles farther down. Where the current of the river meets the slack water of the Gulf of Mexico, bars are formed which require special treatment.

A general view of the conditions has been presented in order to make it plain that the problem involves different treatment in different reaches of the river.



MISSISSIPPI RIVER
VICINITY OF PT. PLEASANT MO.
79 MILES BELOW CAIRO.
1897.
SCALE OF FEET
1000 0 1 2 3000
SOUNDINGS ARE REFERRED TO MEAN LOW WATER, ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.19 FT.
GAUGE AT TIME OF SURVEY = 8.9 FT. OR 6.7 FT. ABOVE MEAN LOW WATER.
ZERO CONTOUR ------ CORRESPONDS TO WATER SURFACE OF MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
PT. PLEASANT
79½ R.
WRECK
U. S. L.
SAND BAR
PHILLIPS LD'G
81 R.
MAIN RIVER BANK
LAZELLES LD'G
82½ R.
MUD

FIG. 1.

Sand Bars as Obstructions to Navigation.—It is not easy to find a satisfactory explanation as to why sediment piles up in ridges instead of being distributed evenly over the bottom. These ridges of sand are usually found on what steamboatmen call crossings ; that is, on the path followed by boats when crossing from a pool lying in a bend along one bank to the pool in the bend of the opposite bank. These bars may be piled up to such an extent that during a high or even medium stage, their crests may be actually several feet higher than the surface of the water at low stage. The thread of the channel at high stages does not follow the low-stage channel, but crosses and re-crosses it.

The first effect of a flood, with its increasing velocity, is to erode the bed and banks and add the material to the load already carried in suspension. This continues until the crest of the flood is reached and the decline sets in. The load is now too heavy for the diminishing velocity and the burden is rapidly deposited and obstructions are formed which, later, prove serious hindrances to navigation. When the river reaches a low stage these act as dams to hold the water in the pools. The slope on the crossing or dam is thereby increased, and likewise the velocity. The crest of the bar consequently cuts out, and if this cutting is confined to one channel a good navigable depth may be the result. If the bar is wide and flat there will probably be several insignificant channels, none of which answer the purposes of navigation. These must be concentrated by contraction or dredging, or both.

Fig. 1 shows rather a complicated case. This is near Point Pleasant, Mo., about 80 miles below the mouth of the Ohio. The general course of the river in this section is straight for several miles. The width is unusually great, due to erosion of both banks. The channel consequently is shallow and shifting. In Fig. 1 the pools are shown by parallel shading, and the figures indicate the depths. The upper pool comes down the right bank and terminates at *A*. The problem then is to reach the pool *B* over the reef which separates the two pools. When that point is reached, navigation to the lower end of the pool *C* is easy. Then comes another crossing to *D*, followed by a narrow broken pool to *I*. Another crossing, and the fourth pool is reached at *J*. Here there is an abundance of water to *M*, then jumping a short reef into the deep pool at *N*, and this bad stretch of naviga-

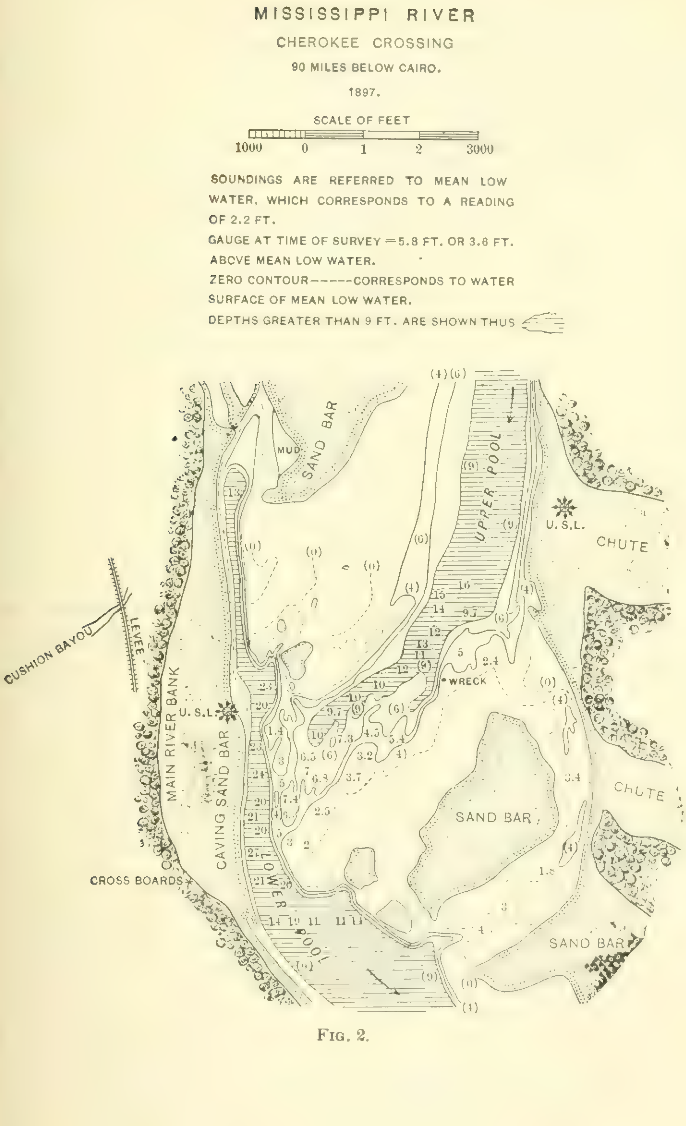

MISSISSIPPI RIVER
CHEROKEE CROSSING
90 MILES BELOW CAIRO.
1897.
SCALE OF FEET
1000 0 1 2 3000
SOUNDINGS ARE REFERRED TO MEAN LOW WATER, WHICH CORRESPONDS TO A READING OF 2.2 FT.
GAUGE AT TIME OF SURVEY = 5.8 FT. OR 3.6 FT. ABOVE MEAN LOW WATER.
ZERO CONTOUR - - - - - CORRESPONDS TO WATER SURFACE OF MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
SAND BAR
MUD
UPPER POOL
U.S.L.
CHUTE
CUSHION BAYOU
LEVEE
MAIN RIVER BANK
U.S.L.
CAVING SAND BAR
WRECK
CHUTE
SAND BAR
CROSS BOARDS
LOWER POOL
SAND BAR

FIG. 2.

tion is passed. Another route of more or less merit might have been found from the pool *B-C* at *L* to the shore pool at *K*, then following this long pool to *G* and over the reef to *H*.

Fig. 2 shows a simpler case and one most frequently met. The problem is simply to cross one reef lying between the upper and lower pools. Having crossed this, the depth is ample for several miles both above and below.

The problem then resolves itself into opening channels through the reefs between the successive pools. As a rule, the pools lie in the bends and the channel line crosses from one bank to the other. During the greater part of the year—say about nine months—the depth in the river below the Ohio is ample for the requirements of navigation. During the other three months navigation is sometimes practically suspended in the third reach on account of shallow water. Whatever is done in the way of opening the reefs at such times answers for one low-water season only. The next flood probably changes the thread of flow to such an extent as to cross the opened channel and obliterate it completely. This is repeated with each high and low stage.

So far, the problem seems simple enough. The only thing necessary is to open a cut through the reef, composed principally of loose sand; but there are other difficulties familiar only to the engineer who has been taught by long observation and experience. While this artificial cutting is going on, Nature is doing some cutting on her own account, but on such a gigantic scale that the work of the largest centrifugal pumps seems insignificant. By the time the engineer gets what he considers a dredge of enormous capacity into position to open a cut from *A* to *B*, Nature perhaps changes her mind and opens a channel of her own down along the shore to *K* (Fig. 1); or, what would be worse, throws such a volume of deposit over the bar *A-B* that the dredged cut is filled as fast as the material can be removed. It is this tremendous volume of material that is moved along the bottom by the current, complicated by unforeseen, subtle changes in direction and force of flow, that often mocks the best efforts of the engineer.

That the difficulties of the problem are not generally understood or appreciated is manifest from the many simple solutions offered by inventors. Men who have never seen the river are prolific in devices for remedying all defects at trifling cost of time or money. They are

by no means alone in the matter, for the men who have spent a lifetime on the river steamers also have ideas, sometimes very good ones, which they have developed and confided to the Patent Office. One inventor appeals to the Secretary of War for some "secret position," where he can study the currents of the river unobserved. He says he has a "nack of inventing," and also "has the nack of getting information." Furthermore, he believes there is some "sculluglery" going on which he might expose. He hesitates about giving his talents to the world, but says he is "willing to confide in the Gov-

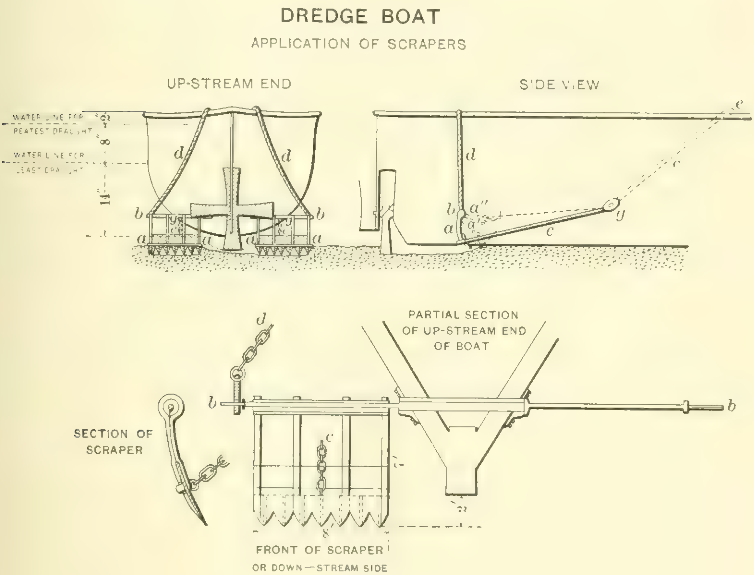


Fig. 3.

ernment." He further says: "Necessity has been the mother of invention with me. Hydraulic engineers may oppose and discourage my project and design, as it is calculated to do away with their work." With his device, "St. Louis would be a port for loading vessels direct from forring ports."

The necessity for some suitable device for the removal of sand bars has long been felt. Some thirty years ago a board of engineers recommended that a prize of $100 000 be offered for the best device for removing obstructing sand bars from navigable streams. Al-

though Congress did not carry out their recommendation by making the necessary appropriation, a great many inventions have appeared from time to time and have been tendered to the Government for use. A description of some of these may be of interest.

These devices can be divided into four general classes, viz., stirring and scraping, current deflectors, jets and the suction dredge.

Stirring and Scraping Devices.

(1) About the earliest application of this principle on the Mississippi was in 1867, when it was decided to improve Pass à Loutre by means of excavating and stirring or harrowing up the minute alluvial material deposited from the heavily laden waters of the river. In this work a double-ended dredge boat, having an excavating screw with four blades 14 ft. in diameter, was used. This screw was similar to an ordinary propeller wheel and was similarly mounted. It was turned by means of a double engine at the rate of 60 revolutions per minute. It reached a depth 2 ft. below the keel. The work of the screw was made more effective by means of an auxiliary scraper attached to the up-stream end of the boat on either side of the keel (Fig. 3). The boat was moved down stream over the bar with the screw operating and the scrapers in position. In this way some of the bar material was again brought into suspension and carried off into deep water by the current.

During the first month's work with this dredge the depth was not materially improved. Later, better success was realized. In a little less than two months the depth was increased from 11 to 17 ft. The chief difficulty seemed to be in weak propeller blades, which were frequently broken and could only be renewed by docking the vessel.

This device was intended to cut out and maintain a 20-ft. channel through the bar at the mouth of the river.

(2) At Southwest Pass the same result was expected from the use of conical screws attached to the bow of a suitable boat. These cones were 20 ft. long and 5 ft. in diameter at their bases. They were set so that their points came together at the boat's stem and their bases were separated so as to cover a width of 20 ft. from out to out. Their axes were horizontal, the salient angle they formed being foremost. The flanges of the screws were 12 ins. wide at the base of the cones and diminished to 6 ins. at the points.

When these enormous screws were put in motion it was very difficult to guide the boat. The material was readily plowed up, but it was not broken sufficiently fine to be carried away by the current.

(3) In 1867 there was appropriated the sum of $96 000 for the construction and operation of two scrapers or dredges on the upper Mississippi, between St. Paul and the mouth of the Illinois River.

The first efforts made to remove sand bars by means of the scrapers invented by Col. Long was in the fall of 1867. These scrapers consisted of a frame attached to the bow of a boat and carrying a heavy cross-bar, to which were attached 6 steel buckets or cutters. This frame could be raised or lowered at will. In operating, the boat went to the upper side of a reef, the scraper was lowered and the boat was then backed slowly down stream, scraping the sand with it down to deep water below the reef. This operation was repeated until the desired depth was obtained.

Two side-wheel steamers, the *Montana* and the *Caffrey*, were equipped with these scrapers by the Government, and, for a time, the steamboat owners operated a scraper boat between Keokuk and St. Louis at their own expense.

The *Montana* was 210 ft. long, 35 ft. beam and 5½ ft. depth of hold. She was equipped with two engines having 20-in. cylinders and 7-ft. stroke. The *Caffrey* was 150 ft. long, 30 ft. wide, 4½ ft. hold, with a draught of 32 ins. She had 15-in. cylinders with 5-ft. stroke.

This last boat was equipped and ready for work in October, 1867. Her first work was on a bar near Gray Cloud, 17 miles below St. Paul. Only 3½ ft. could be carried over this bar, and the regular packets could not cross it. After about four hours' work with the scraper the depth was increased to 3½ ft. entirely across the bar. The scraping was continued for two days and a depth of 4 ft. was secured. By November 15th all the bars between St. Paul and Prescott had been scraped and the depths increased to 3½ or 4 ft. At that date the packet companies notified the engineer in charge that the scraping had removed all obstructing bars and that no more work was required.

In 1868, when navigation became difficult. the scrapers were put into commission and worked throughout the season. They succeeded in deepening the bars from 8 to 18 ins., and this was generally accomplished with a few hours' work. Beef Slough was deepened from 3½ ft. to 4½ ft. in 35 minutes.

On the whole, the results were so satisfactory that steamboat owners announced that their boats had been making regular trips without interruption, "a condition of affairs never before known at this stage of river in the experience of pilots of 35 years' standing." The largest steamers had been able to reach St. Paul in the low-water season during two successive years, when without the aid of the scrapers they would have been obliged to tie up. Steamboat men said:

"These scrapers can be relied on to increase the depth over the crests of the bars at low water from 8 to 12 ins. This nearly doubles the carrying capacity of the boats now in use."

This scraping was continued for several years at a cost of about $20 000 per annum for each steamer, but as the relief was only temporary and had to be repeated from year to year, it finally gave place to the so-called permanent improvement, consisting mainly of channel contraction.

It should be borne in mind that in the portion of the river where the above-described scrapers were used, the obstructing sand reefs are quite short.

(4) An "excavator for water courses having current" consists of a horizontal shaft attached to the stern of a suitable boat, and fitted with a series of serrated metallic disks 40 ins. in diameter. Each disk has 29 cutters or teeth, which are turned alternately in opposite directions. These disks are revolved by the necessary machinery, at the rate of 100 revolutions per minute. The bar to be removed is cut out by running back and forth until the required depth is obtained. It is claimed that this device "is found highly efficient in raising deposits in channels and separating the earth into small particles so that it may be carried off by the current," and that the amount of work it can accomplish in a given time is "equal to that of 249 common bucket-dredging machines."

It is well to note that these claims are not based on actual trial in practical work.

(5) A "submarine plow," attached by means of timbers to the sides of a boat, aims to plow out a furrow in the bar. Spiral springs are attached to the timbers holding the plow so as to force it down to its work, and so that it will adapt itself to any irregularities of the bottom. The efficiency of the plow is increased by water-jets forced into the furrow for the purpose of breaking up and scattering the deposit

moved by the plow. Beyond this, the current is relied on to carry the deposit to deep water.

(6) A "channeler," consisting of a horizontal revolving drum made of boiler iron, with a series of plows and scrapers attached to its convex surface, is conspicuous among the earlier inventions. The revolving drum is 40 ft. long, and is suspended by journals near its extremities. It is revolved by means of two 35-H.-P. engines, which operate the drum through gear wheels acting on a large cog wheel encircling the drum half way between the ends.

The scrapers are set spirally on the drum for the purpose of carrying the material outwardly each way from the middle. When the material reaches the end of the drum it is supposed that the current will take it up and carry it away, leaving a clear cut equal to the length of the drum. The drum and its shaft are attached to the stern of a boat. In this case the attachment is rigid, and the depth is adjusted by means of water-tight compartments in the boat which can be filled and pumped out at will.

In operating this device the boat is dropped down on a bar with bow end upstream. The drum is then made to revolve as the boat moves down on the reef. By means of anchors and lines attached to windlasses, the boat is drawn stern foremost down across the bar as rapidly as the scrapers on the drum loosen up and remove the deposit. It was supposed that the excavated material would be thrown from the ends of the drum with sufficient force to carry it entirely free from the cut, and it was estimated that in sand this device could make a cut 40 ft. wide and 3 ft. deep at the rate of 150 ft. per hour.

In moving from one bar to another the drum is used to propel the boat, which in this way, it was claimed, could travel at the rate of 8 miles per hour. The estimated cost of a craft of this kind was $25 000. A crew of 12 men was required to operate it.

Like many others, the merits of this device were never tested by actual work on sand bars.

(7) Another "scraper" consists of two arms, each 80 ft. long, placed parallel to each other and hinged at their middle points on the stern overhang of a side-wheel steamer. At the outer extremities of these arms is fastened a cross-beam, to which is hinged the scraper backing, which is rectangular in shape and made of boiler iron properly stiffened. Several scoop-shaped scrapers and plows, placed

alternately, are attached to the lower edge of the backing. From each end of this cutter a chain leads to the steamer for the purpose of taking up the strain and regulating the inclination of the scrapers. The scraper is raised or lowered by tackle leading from the inboard end of the arms to a drum on the deck of the steamer.

With this scraper the boat moves over the bar from its up-stream side and with head down stream. The sand is thus scraped down to the deep water. The operation is repeated until sufficient depth is obtained.

(8) "A steam circular sand bar dredger" was brought to public notice in 1878. This machine commended itself on "account of its practicability." With it, the inventor says:

"There is no need of costly engineering and surveys, no examinations to be made and no commissioners to report on what is necessary to be done. As a sand bar dredger it is invaluable; commencing at the lower edge of a sand bar by lowering the wheel to the required depth, a channel may be cut through in a remarkably short space of time, the current of the river carrying away the sand displaced by the revolving toothed wheel."

This machine was also intended for opening channels through ice. It was claimed that it could cut its way through solid ice 2 to 3 ft. thick at the rate of 4 to 6 miles an hour, and thus open a channel wide enough for steamboats.

"The harbors of the world heretofore closed by ice from 3 to 5 months in the year can be kept open so that ships can go to sea every day in the year." "One of these boats on each canal level would keep them open to commerce during the winter."

Offers were made to "guarantee" 10 ft. of water in the channel of the Mississippi River from St. Louis to Cairo in the driest season, with one "dredger" costing $40 000.

"The advantages of this invention to the commerce of the world is beyond computation."

(9) Another scraping device consists of a steel hull with a wedge-shaped prow, with auxiliary wings attached. This was intended to act something like a huge snow-plow, the wings or deflectors being designed to carry the loosened material far out each way from the cut. Jets were also provided to loosen up the material.

In actual operation the hull is sunk to the bottom by admitting water, and is then pulled up stream by powerful towboats. The wedge-

shaped prow buries itself in the sand, which is pushed along as the boat is moved, and finally finds its way out to the ends of the deflectors. In moving from one bar to another, the boat is first floated by pumping the water out. This is a comparatively new device.

(10) Another comparatively recent device consists of a gigantic harrow having a water-jet at the extremity of each tooth. The pressure chamber forms the frame of the harrow. This harrow was to be attached to the bow of a boat and be provided with suitable rigging and winches by means of which it could be raised or lowered at will.

(11) A different design, or, as the inventor calls it, "Means for deepening the channels of rivers," consists of four "circular plows," or agitators, revolving on vertical shafts passing through the hull and operated by the necessary driving engines and gearing. Each screw can be raised or lowered independently, as required. These screws are intended to loosen up the material, and the natural current is supposed to do the rest. The forward pair of screws are smaller than the others, for the purpose of first opening a small channel, which is widened by the two which follow. The boat is moved back and forth until the required depth is reached.

It was claimed that four boats equipped in this way could make and keep open a channel 10 or 12 ft. deep between St. Louis and Vicksburg.

(12) An "excavator," similar to that described in Section 10, is attached to the bow of a boat, and has vertical shafts with scrapers at the lower end which are revolved by means of belts. The depth of the shafts is regulated by a screw at the upper end of each. This device also relies on the current to carry off the loosened material. The inventor says that "The revolving cutters cut up the material and churn and stir it to a considerable extent, so that it will be carried away by the current."

(13) Another "channel plow" inventor has a device which he claims will produce

"A mean smooth bottom, circular in form and non-resisting, which will be of sufficient width and depth to contain and carry off within banks the greatest floods, resulting in the disappearance of all snags, sawyers and bars."

It will "Provide a steady flow of tranquil waters at all seasons and stages." It will cause a "Total cessation of overflows and the consequent reclamation of many river-bottom lands, and that, too, without the aid of levees."

This, certainly, covers the whole ground. The earnest efforts of all hydraulic engineers are directed toward the accomplishment of the results set forth, but none of them are as confident of success as the inventor seems to be.

(14) One of the latest devices offered, to solve the problem of removing sand bars, is made in the form of a conveyor such as used in mills and elevators, but, of course, much larger in size. This spiral or screw has blades several feet wide attached to a hollow shaft, and is made up in sections 10 ft. in length. These sections are strung on a steel spindle until the whole screw is of sufficient length to reach across the bar. To utilize this device, it is dropped down over the bar, and, when in proper position, it is held in place by means of anchors or piles connected with wire cable to the upper end of the spindle. It, of course, trails down stream parallel to the current, and the pressure of the current against the blades causes the screws to rotate. In this way the sand is loosened up and conveyed to the pool below the bar.

All the power required in this case is derived from the current, and as the screws themselves are comparatively inexpensive, this would be a very economical method of cutting down bars if the inventor's ideas of efficiency were realized.

(15) A simple arrangement, intended to make each boat independent of outside aid when shoal water is encountered, is that in which the hull of the boat is provided with wings, which, when not in use, fold closely against the sides. On approaching a shoal the wings are opened and a portable dam is thus formed which raises the water; and this, in addition to the pressure on the wings, enables the boat to ride safely over what would otherwise be an impassible barrier.

(16) The "automatic sand dredger" consists of a stern-wheel boat to which is attached a horizontal shaft bearing a number of cutting disks, constituting a dredging wheel. The extremities of this wheel shaft are connected by bars to a shaft extending across decks near the bow of the boat. A convenient hoisting drum serves to raise and lower the cutter. The cutter has considerable weight, and the hooked cutters penetrate the sand, which is loosened and lifted up where the current catches it. As the boat is moved over the bar with the cutter lowered to the bottom, the cutter wheels roll along the bottom.

(17) Another scraping device consists of a suitable boat provided with two long parallel booms or spars across decks about amidships.

These booms project about 50 ft. out from the sides of the boat. The ends of the booms are braced and are supported by guy lines leading to a central mast, and also a second pair of guys leading to the bow and stern of the boat. The wings are hinged on each side of the boat to the forward boom. They extend from the side of the boat out to the end of the boom, and are wide enough to reach the bottom when let down on a bar which it is desired to deepen. The lower edge of these wings is weighted and provided with teeth or plows which penetrate and stir up the material. Guy lines, leading from the lower edge of the wing to winches on deck, serve to hold the teeth in contact with the bottom. When not in operation the wings are lifted up flush with the deck by means of tackle attached to the rear boom.

In operating this device, the boat moves down stream over the bar, carrying the sand with it over the reef. The current acting on the wings helps the boat along. This operation is repeated until the desired depth is secured.

(18) Another device is a "dredging machine" for use in "working out the channels of rivers and in removing sedimentary formations from the beds of running streams." The agitators are called "wipers." These agitators are attached to the stern of a side-wheel boat and are operated by means of bevel gears so arranged as to revolve the wipers in opposite directions. They are hung on a cross-shaft which admits of their being raised or lowered. The wipers are made with spiral flukes, so that the lower points will enter the deposit first and then draw it upward, thus lifting it from the bottom, and disintegrating it so that the current can carry it away. The number of agitators will depend on the magnitude of the work. The boat is to be moved back and forth over the bar, with the agitators at work, until the required depth has been obtained.

(19) A ship's drag for dredging rivers is a device to be attached to a steamboat or barge and dragged over the bar to be deepened. The teeth rake and stir up the bottom to such an extent that the current carries the material away. The teeth of the drag are rounded in front and have a groove in the back. The object of this groove is to form an eddy and thus aid in keeping the matter in suspension until the current takes it up. The frame holding the teeth can be separated into several jointed parts by means of hinges, and in this way the scraper will conform to the shape of the ground over which it moves.

Current Deflectors.

(1) The most pretentious of current deflectors is a "Machine for Deepening River Channels," invented in 1879. The object of this machine is to "deflect the current of a river downward, and thus cause the said current to deepen the channel." This consists of a boat to which is attached a triangular box which can be lowered to a point near the bed of the stream, the axis of the box being at right angles to the boat and to the current. The up-stream edge of the box has flanges for regulating the flow.

In operating this machine it is moved across the bar, either up or down stream. The current strikes against the inclined side of the box and is thereby deflected downward, which causes it to impinge against the bed of the river and wash away the sand to a considerable depth. In case the bed is composed of clay not easily abraded, it is first loosened by means of a revolving toothed cylinder, and then carried away by the deflected current. The boat is held in position against the current by means of arms pivoted to the boat, the lower ends resting on the bottom. A wheel in front of the device is of sufficient diameter to show above water when resting on the bottom. The object of the wheel is to bring up samples of the bottom of the river so that the "engineer may know the exact kind of bottom there is at each point," and conduct his operations accordingly.

The device is further provided with a graduated disk attached rigidly to the arms supporting the triangular box. To the center of the disk is pivoted a weighted pointer which will always hang vertical and hence show the inclination of the box supports and consequently the depth of water. The inventor also provided an automatic recording device which would plat the profile of the bottom as the boat passed over it.

In moving from one bar to another the deflecting box is to be raised above the surface of the water by means of lines attached to the rear arms. Just how this is to be accomplished is not clear.

(2) Another device for deflecting the current downward on the bed of the stream is styled a "dredger," and is described as a "simple, inexpensive and efficient device for deepening the channels of rivers, removing sand or mud therefrom and for preventing the formation of sand bars." These desirable results are accomplished by means of several parallel plates of different lengths which are set at an angle

with the vertical, but having the lower edge of the plates in the same plane. These plates are securely held in a suitable frame, and the whole is supported on short legs.

This frame, with the attached plates, is set on the bottom of the stream with the plates inclined up stream. The current, in striking the plates, is deflected downward and thus produces a scouring effect on the bottom. When sufficient depth has been secured it is moved to a new position. The size and number of these devices to be used is, of course, governed by the magnitude of the channel desired. The device is to be handled from suitable boats or barges. A device similar to this was used in the improvement of the South Pass with some success.

If it is desired to erode a bank the device is turned on edge so as to throw the current against the bank. To prevent the formation of sand bars, this inventor says that all that is needed is to suspend near the bottom V-shaped plates with perforations at the angle. The current caught by the open arms of the plate will escape through the perforations with force enough to keep the sand on the move.

(3) Another "device for regulating the flow of streams," deepening the channels and removing obstructions is also intended to deflect the current downward against the bottom. It consists of a frame having a suitable number of cross-bars to divide the interior space into cells of proper size, depending on the magnitude of the stream. The upper surface of the frame and cross-bars are filled with teeth, which, together with the cells, obstruct the even flow of the water and produce a "violent suction or eddy in the current." This increases the momentum or gravity of the water and causes it to "force itself violently" through the orifices between the cross-bars and thus churn up and wash away the bed of the river. In this way the sediment composing the bottom is agitated to such an extent that the stream is compelled to take it up and carry it away in suspension.

The device can be made in any number of sections joined together by chains and anchored to the bottom of the river. In this way, it is said, bars can be prevented from forming and the bed of the river can be depressed to any desired extent.

It is perhaps well to note here that the principle involved in the foregoing devices, and which is relied upon to erode the bottom, is to be found in several other devices where it is expected to induce deposit.

If the current can be deflected downward and erosion produced, then a barge anchored over a reef should produce similar results, as the current striking the end rake would be deflected downward. In practice, it is a well-known fact that a barge anchored in shallow water will very soon cause the sediment to deposit. In dredging, the plant is never left in the cut for any great length of time when the pumps are not in operation. If a break-down occurs which necessitates extensive repairs, the plant is swung out of the cut so that the current may have free access to it.

Nearly all the current deflectors proposed are intended to deflect the current laterally, and for this purpose inventors have brought out jetties of various kinds.

(4) A simple and efficient jetty deflector is made by driving piling along the desired line at intervals of about 20 ft. On the up-stream side of the piling small flat boats 16 by 40 ft. in size are lashed in a continuous row. Corrugated steel aprons 10 x 10 ft. in size are hung over the upper side of the flats at a considerable angle with the river bed, to allow for straightening up as the bottom scours out. The corrugated sheets of steel are stiffened by three angle bars of 3 x 3-in. section firmly riveted to each sheet. The jetty is also constructed without the boats by attaching to the piling a stringer on which the plates are hung.

This jetty has been used with success between the Missouri and Ohio Rivers. Twelve hundred feet of jetty was built in one locality in four days with a force of twenty-two men. It cost about $2 per running foot. The effect was almost immediate. The water was deepened from 4 ft. to 7 ft. with a fall of 1 ft. in the stage of the river.*

A similar jetty has been proposed with the substitution of canvas for the steel plates and omitting the use of the flats. In this device it is proposed to make a curtain of canvas, say 15 ft. wide and 100 ft. long; one edge and the seams to be reinforced by sewing in hemp rope of suitable size with good lashings at the edge. At the other edge a chain is to be lashed to the canvas as a sinker. The whole is rolled up like a reefed sail. In order to use it, the piling is driven as before. The roll of canvas is stretched across the up-stream side of the piling above water and lashed at each pile. When everything is ready the reef lashings are to be cut and the weighted edge causes the canvas to unroll

* Report of Chief of Engineers, U. S. A., for 1896.

and sink to the bottom. The flexibility of the chain insures a close contact with the bottom. As many lengths of curtain can be used as may be found necessary.

(5) Similar deflectors are made portable by means of double-hull boats which admit of dropping sheet-piling down between the hulls. When it is desired to deflect the current, or cut off a side chute, a number of these boats are placed in position, end to end, until sufficient length is obtained. These are known as the King jetties.

(6) Another way of accomplishing the same result is by means of steel barges of suitable length, known as the Marsh jetties, which are designed to sink by filling with water. The ends are wedge-shaped and the sides are straight, so that one boat may be lapped and fastened to the next, and so on until sufficient length has been obtained. In this way the current will be effectually deflected to the channel which requires deepening.

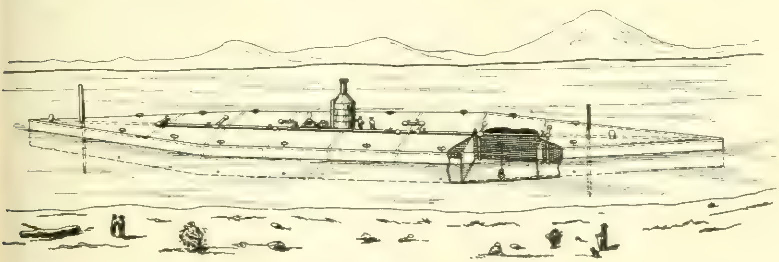


FIG. 4.

When the desired results have been obtained the water is pumped out of the hulls, which are thus floated and are then ready for another bar (see Fig. 4).

(7) Another inventor, somewhat earlier than the last named, hit upon a wooden sinkable deflector and plow, by means of which the "Sand, sediment, etc., at the bottom of the river will be sufficiently irritated to enable the current to carry it off." The jetties consist of strong wooden boats triangular in cross-section, the apex of the triangle forming the keel. These boats are designed to be from 500 to 600 ft. long. The plan would also be triangular, the greatest width being in the middle and then tapering off toward each end, which terminates in a strong timber head. The upper portion is open, with occasional cross-bracing. In order to use this jetty it is towed to the point desired and then sunk by filling it with water.

In order to secure the required length several may be sunk with a slight overlap. In this way the current is supposed to be deflected so as to cut out bars, or even make a cut-off if required. When the work is done the jetties are floated by pumping out the water. It is claimed that they can be "raised and removed in two hours."

An attachment termed a plow is applied to the towboat which handles the jetties. Its function is to loosen up the hard material so that the current from the jetties can get hold of it and carry it away. This plow can be raised or lowered at will, by suitable winches which are provided for that work. It is claimed that by the use of these devices rivers can be plowed to a depth of 30 ft.

(8) Some inventors have directed their energies toward the use of material at hand. One of the earliest of this kind of deflector proposed was made of huge trees with branches intact, and sunk in the desired place by means of heavy weights attached to the butts, while the tops were free to take the direction of the current. A row of such trees are sunk side by side, and, if need be, others are added on top. This is not intended to stop the flow to such an extent as to form eddies, but sufficient to silt up the stream at or below the dam and deflect the water to the desired channel, and cut out the bar against which the current is directed.

(9) Another inventor uses similar trees in a different manner, which he claims "Is founded on natural and therefore correct principles, to correct, widen, deepen and confine the channel of the rivers."

In this system the trees are sunk as before, but the tops must be placed up stream to catch the small drift and sand, which very soon accumulates and fills up the dike. The trees may be held in place with piling. This jetty is intended to induce rapid deposit, which soon forms around the trunks of the trees, "Firmly anchoring them to the bottom ; the branches are next surrounded and covered by sand, completing by a natural process the indestructible and impregnable dam."

Water-Jets and Auxiliary Devices.

Several jet devices have been proposed which aim to make each steamboat independent of any general improvement of the channel by providing suitable jets which will enable the boat to work its own way through the bars.

(1) One of these jet devices consists of two double-acting pumps which force water through 28 3-in. nozzles arranged symmetrically about the bow of the boat. On reaching a shallow bar, the force pumps are started, the jet valves are opened, and the boat is pushed ahead as the sand yields to the eroding force of the jets. In this way the boat carries a pool with her across the reef, and at the same time will probably improve the depth for the boat that follows.

(2) Another device aimed to accomplish the same result with a different kind of pump and a different arrangement of the jets. It was claimed for this invention that its use would make channel improvements with dredges, contraction or other means entirely unnecessary. By this scheme each steamboat was to be equipped with a centrifugal pump with a capacity of about 2 000 galls. per minute. The discharge pipe was designed to run the whole length of the hull along the center truss. The jets emanating from this pipe were arranged to discharge along the lower line of the hull directly on the sand. The jet nozzles were arranged so that they could direct the stream either forward or backward, so as to always act with the current, whether the boat were going up stream or down.

When the boat reached a bar the pump was to be started and the jets opened, the boat being constantly forced ahead until finally carried over the reef. The estimated cost of this attachment was only $3 000 for each boat, while the cost of delays in a single trip caused by grounding might readily exceed this amount.

In urging the utility of this method the inventor makes the following statement :

"It cannot be expected that the Government will every year spend thousands of dollars to remove sand bars which re-form at every flood. Those interested in river navigation ought to make themselves independent of such obstructions and of Government aid, but they will not entertain the idea until forced to."

Admitting that these devices would enable a single boat to force its way through obstructing sand bars, there still remains the great bulk of traffic which moves in barges, that cannot be carried over by the methods proposed.

(3) Another jet device, intended to remove the obstructing bar, is called a "Submarine plow or hydraulic apparatus for removing sand bars." It consists of several jet pipes fixed to a drag frame which is

pivoted to the bow or stern of a suitable boat. Each pipe terminates in three jets, one of which points downward and loosens the material, while the other two are bent so as to pick up the loosened material and carry it upward where the current will take it and move it along. The combined force of the current and the jets carries the particles of sand some distance before they come to rest. The curve of each jet pipe is filled in with a web, so that if it strikes a log or other obstruction it will slide over it without catching. The whole device is provided with tackle and winches conveniently arranged to raise or lower the jets, and suitable pressure pumps supply the necessary water.

It is claimed that a vessel with a 500-H.-P. equipment of this kind can erode a depth of 5 ft., taking out a cross-section of 27 sq. ft., at the rate of 6 miles an hour, throwing the sand and gravel to the surface in 30 ft. of water.

This device was used in Gedney's Channel, New York Harbor, but was abandoned on account of the slack current at ebb tide and the influx of material at flood tide.

(4) Water-jets were used with marked success on Horse Tail Bar, near St. Louis, in 1881. Four pile-drivers were lashed two abreast with their heads together. Each one of these drivers was equipped with a Worthington duplex pump with a 7½-in. steam cylinder and a 4½-in. water cylinder having a 10-in. stroke. The capacity was about 165 galls. per minute each. The four jets were brought together. Anchors were laid and the drivers were pulled back and forth across the reef with a steam windlass, all the pumps working in the meantime to their full capacity. In ten hours' actual work in this way the channel was deepened from 6 ft. to 8.3 ft. for a width ample to pass the largest tows.

(5) In 1896 a jet dredge was constructed under the supervision of Major Thos. H. Handbury, for the work between St. Louis and Cairo. The main features are two 15-in. centrifugal pumps driven by direct-connected 15 x 14-in. engines. The steam is furnished by two 4-flue boilers 28 ft. long with 42-in. shells. This machinery is mounted on a suitable barge.

The suctions lead over the sides of the barge, and the discharge leads over the bow. The capacity is 20 000 galls. per minute. This water is forced, through four 12-in. discharge pipes with flattened nozzles, against the sand, which is thus stirred up and washed into deep

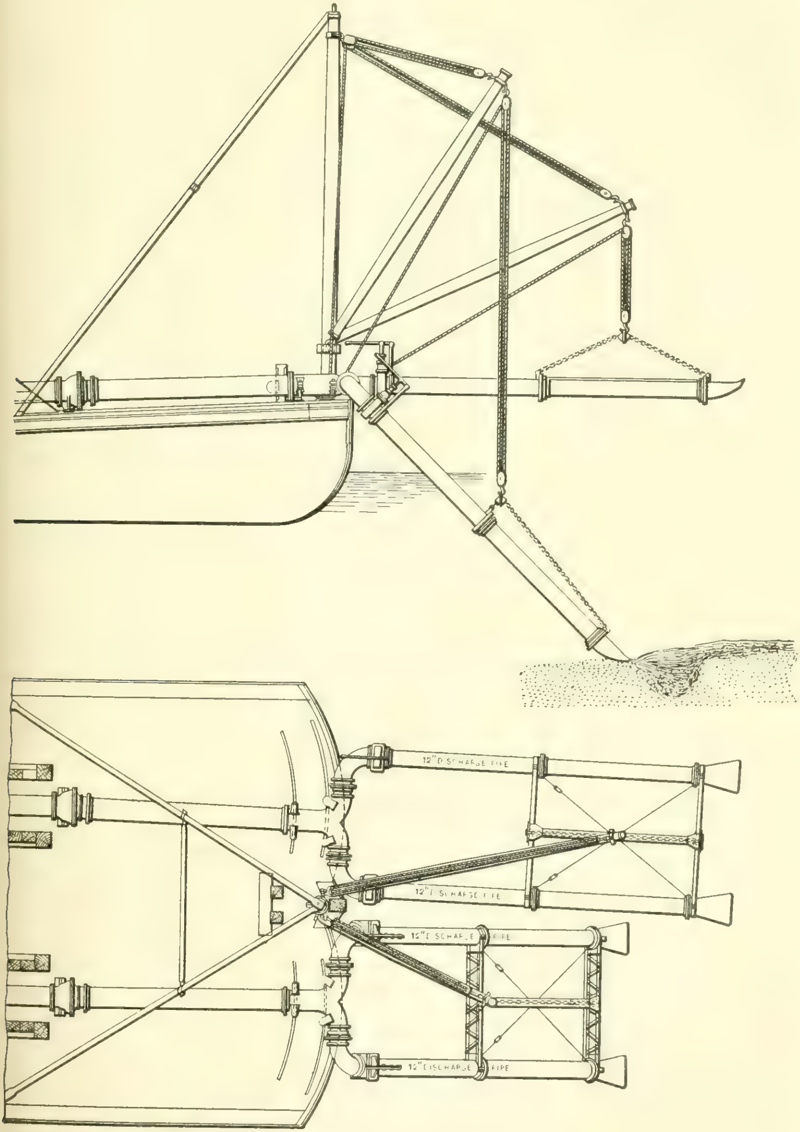


Fig. 5.

water. The attack is from the up-stream side of the bar, and the boat is dropped down, with jets in operation, until the lower pool is reached. This operation is repeated until the desired depth is obtained. The plan and elevation of the bow of this jet dredge is shown in Fig. 5.

On short reefs, this device is reported to be quite efficient. Where the reefs are of considerable length the accumulation of sand in front of the jets becomes too great to be moved economically in this way.

(6) By far the most pretentious improvement scheme by means of jets, yet proposed, is the "Adams Flume," the construction of which was authorized by Congress in 1879. The purpose of this device was to "Establish a permanent channel in the Mississippi River from St. Paul to the Gulf of Mexico," a distance of some 2 000 miles.

It consisted of a triangular iron tube with sides of about 12 ins. The different lengths of this pipe were provided with flanges at both ends, so that they could be bolted together, a rubber gasket being placed at each joint to give the pipe sufficient flexibility to adapt itself to the undulations of the bottom and the curvature of the channel. The pipe was divided longitudinally into two compartments by means of a metal partition. The lower compartment was provided with ½-in. jets at intervals of 1 ft. Clear water was pumped into the upper compartment under a pressure of 5 to 8 lbs. per square inch. By means of suitable valves the water could be let into the lower compartment and through the jet pipes; then, in the language of the inventor:

"It stirs up the sand, holding it in suspension till the current carries it off to some low place where it forms banks like a canal. The number of jets to be opened at any time is governed by the pressure. Where there is any obstruction it is opened on like a battery; no sand, gravel or moderate hard-pan can stand near it. To lay this pipe in one unbroken line in the center of the river from its head to the Gulf, it will scour a channel the necessary depth and width throughout and keep it open all through."

In this work one small Worthington pump was considered sufficient for each 100 miles of pipe. Clear water was said to have greater eroding power than water containing sediment, and in order to secure it, the river water was allowed to flow into a tank or well attached to the boat and the pump took its supply from the well. Just how the sediment was to be eliminated by this process is not altogether plain.

More than eight years and about $40 000 were spent in attempts to prove the practicability of this device, but the project was finally abandoned without ever laying a foot of pipe in the river channel. This large waste of time and money cannot be charged to the engineers, for Congress, through the Secretary of War, dealt directly with the inventor, the law requiring that the device should be made and tested under his supervision and direction. For five years the "preliminary arrangements" were going on, the expenditures having reached the sum of $27 500. One locality after another was agreed upon as suitable for the trial, but there was always something to prevent an actual test. Another appropriation of $15 000 was made, and the "preparations" were continued for several years more without results of any kind. A board of engineers was then appointed to examine and report on the advisability of continuing the experiments. Their conclusions were that it could not be successful even on a small scale.

The engineer officer in charge of the district was instructed to take charge of the work pertaining to the flume. The only thing found to show for the $40 000 was a small amount of sheet-iron pipe made for experimental purposes, and little or none of this was completed and ready for use.*

(7) The removal of bars by means of blasting has been tried, but with unsatisfactory results. Torpedoes, loaded with 75 to 160 lbs. of powder, have been exploded at depths of 10 to 20 ft. below the surface of the sand, but the results have invariably proved disappointing. Large quantities of powder in tin canisters laid on the crest of the bar have been exploded without materially improving the channel depth.

In the foregoing pages the author has attempted to give a brief account of the various devices which have been offered to solve the difficult problem of improving the low-water navigation of the Mississippi River. A few of the devices have been actually used, and still fewer have been moderately successful.

The Long scrapers used on the upper Mississippi some 28 years ago were pronounced satisfactory, although the best they could do was to increase the depth over short reefs 12 to 18 ins. About this time the use of temporary expedients gave place to a system of per-

* Report of Chief of Engineers, U. S. A., 1888, p. 1488.

manent improvements, consisting of the closing of chutes, the contraction of wide places by means of jetties or dikes, and the revetment of banks. The inauguration of this policy, to the exclusion of temporary expedients for channel improvement, was evidently a mistake, due probably to the fact that the length of time required by the so-called permanent method before tangible results could be realized was very much underestimated.

In the meantime, competition of railway lines along the banks of the river, and difficulties of navigation, threatened to annihilate the river traffic. Steamboat men saw their profits dwindling, their business gradually dying out, and they urgently appealed for relief. They could not wait for the completion of permanent improvements. Immediate temporary relief must be found, or the river as a great commercial highway must be abandoned. It was in response to this appeal that temporary channel improvements were again taken up and were finally, after several years of agitation, made obligatory by Congress, for the lower river, in the River and Harbor Bill of June 3d, 1896.

In the expedients which were tried or suggested for temporary improvement of the channel, and which have been described in the preceding pages, the chief defect lay in relying on the current to carry off the material after it was loosened up. It was generally thought that if the crest of a bar was thoroughly loosened up, the current would carry the material away, and thereby the depth would be increased. Engineers very soon saw that but little additional depth could be gained in this way.

River men still talk of a "crust" on a sand bar, and believe that it prevents erosion by the current until broken by scrapers or other means, and that after this is done erosion is rapid and effective. One of the members of this Society was probably among the first to recognize the necessity of carrying the spoil out of the channel by artificial means. In 1870, after experimenting with dipper dredges for some time, Robert E. McMath, M. Am. Soc. C. E., reached the conclusion that

"The successful use of sand pumps and hydraulic conveyors and distributors of débris inclines me to the opinion that a combination is possibly better adapted to the work of river improvement than any means yet used."

In 1883 a plant was provided for dredging in the Mississippi River above the mouth of the Illinois River. This plant consisted of an

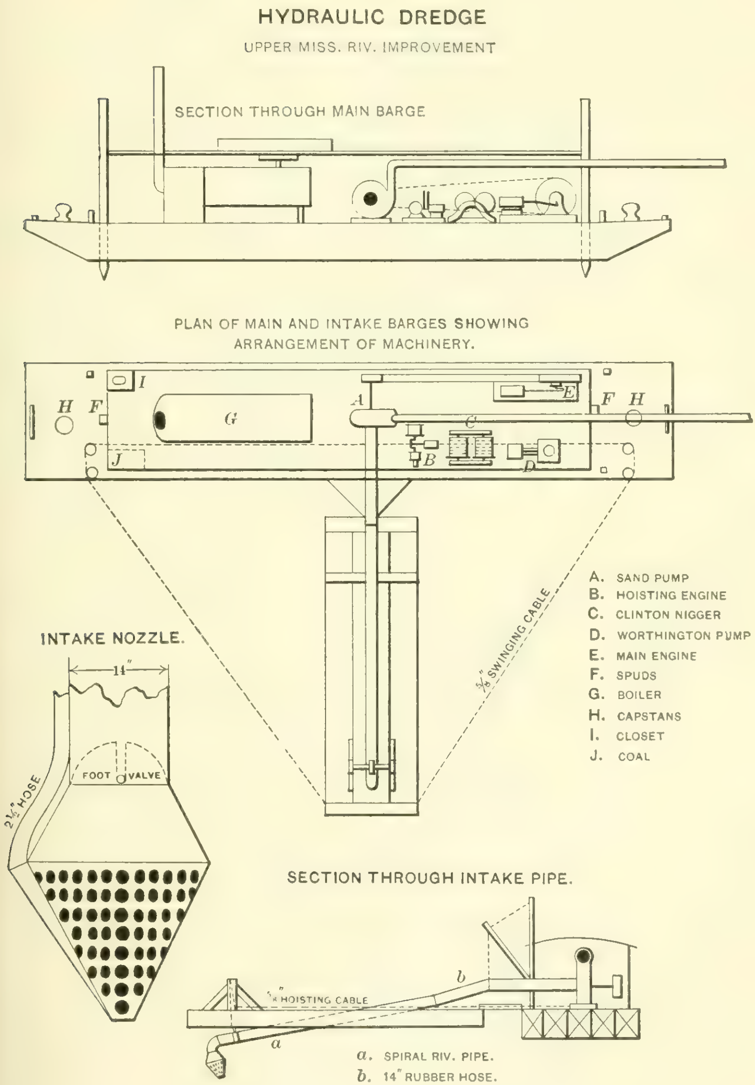

HYDRAULIC DREDGE
UPPER MISS. RIV. IMPROVEMENT
SECTION THROUGH MAIN BARGE
PLAN OF MAIN AND INTAKE BARGES SHOWING
ARRANGEMENT OF MACHINERY.
A. SAND PUMP
B. HOISTING ENGINE
C. CLINTON NIGGER
D. WORTHINGTON PUMP
E. MAIN ENGINE
F. SPUDS
G. BOILER
H. CAPSTANS
I. CLOSET
J. COAL
5/8" SWINGING CABLE
INTAKE NOZZLE.
14"
FOOT VALVE
2½" HOSE
SECTION THROUGH INTAKE PIPE.
HOISTING CABLE
a. SPIRAL RIV. PIPE.
b. 14" RUBBER HOSE.

Fig. 6.

Osgood dipper-dredge and six dump-scows, the whole plant costing $29 348. It was used with good results. The first work was done at Howard's Bar, near La Grange, Mo., where a channel was opened 1 000 ft. long, 110 ft. wide and 5 ft. deep. It was completed in about a week, and the channel remained open throughout the season. Part of the material was removed by dump-scows, but the greater part of it was dropped beside the cut, which was located parallel to the thread of the current.

This plant was also used at other places with good results. Major Ruffner, the engineer officer in charge, in the following words, recognized the utility of dredging :*

"The experience of the past season has shown that much assistance can be given to navigation by this kind of work, which, though temporary in a certain sense, is likely to be permanent for one season if not longer. * * * By the temporary use of dredges, at an expense so comparatively small as to be fully justifiable, I have no doubt a good navigable channel can be maintained during the low-water season on this stretch of the river, and if the navigation of the river is to be continued with success by steamboats, such temporary aid must be given until such time as the permanent channel improvement is completed."

In 1887 a contract was entered into for a hydraulic dredge and decked flat-boats to support 500 ft. of discharge pipe, to be used in the district above named. Experience had shown that a dipper dredge could do good work under favorable conditions, but for opening temporary channels through bars it was found to be too slow in its operations and hence too expensive. A dredge provided with a centrifugal pump for pumping sand and water from the channel and delivering the same through pipes far enough away to clear the channel was manifestly an improvement on the old method with bucket-dredges and dump-scows (see Fig. 6). This hydraulic dredge was equipped with a Van Wie No. 12-pump with a 14-in. intake and a 12-in. discharge. The pump was run at the rate of 330 revolutions per minute and the material was lifted about 9 ft. above the surface of the water and discharged from 300 to 800 ft. from the pump. The intake nozzle was provided with a jet supplied by a Worthington pump, to aid in stirring up the material so that the suction could take hold of it.

The steam plant consisted of one boiler 22 ft. long, and having a 42-in. shell with ten 6-in. flues. The main engine was of the Wright

* Report of Chief of Engineers, U. S. A., 1884, pp. 1561–1567.

and Adams pattern, 14-in. cylinder and 18-in. stroke. There was also a hoisting engine and a "nigger engine," each 6 x 8-in. double cylinders.

The intake pipe is supported by a double pontoon, or two barges 4 by 50 ft., framed together with a 4-ft. space between, which stands at right angles to the dredge when in operation. The dredge end of this pontoon is pivoted to a triangular brace attached to the dredge, and the outer corners of the pontoon are connected with the dredge by means of wire cable-guys, passing through sheaves to the "nigger engine," which serve to swing the pontoon, with the suction, around in a semi-circle with a radius of 48 ft. A hoisting cable serves to raise and lower the suction between the two parts of the pontoon. The intake pipe has a 14-ft. length of rubber that connects the iron pipe of the suction with the iron pipe from the dredge and gives the pipe sufficient flexibility. All engines are located so as to be handled by one man.

The machinery is mounted on a wooden hull 20 ft. wide, 100 ft. long and 4 ft. deep, and is housed in by a suitable cabin. There is a spud 12 ins. square at each end of the dredge which serves to hold it in the desired position. There are a number of small flat-boats 12 by 32 by 2 ft., each of which carries 45 ft. of discharge pipe. The several lengths of pipe are connected together with rubber hose. The flat-boats are coupled together by triangular wooden frames bolted to the decks and so arranged as to couple with an iron pin midway between the boats. The discharge pipes are carried on a frame erected so as to give a slope of about 1 ft. in 100 ft.

This dredge is manipulated by means of two head lines anchored well up stream, and the dredge is moved and kept on line by swinging first on one spud and then on the other, alternately. The work can be done either with or against the current, each cut being about 90 ft. in width. The dredge is moved about 8 ft. at a time. A vacuum gauge at the suction side of the pump and a pressure gauge attached to the discharge side serve to show the operator whether the pipes are obstructed or whether the proper proportion of sand is being carried. This dredge delivers from 3 to 7% of sand through 600 ft. of discharge pipe at the rate of 70 to 100 cu. yds. per hour. It requires a crew of 7 men, costing $355 per month, to operate the dredge. The cost for 90 working days, including delays of all kinds, and the current

repairs, was $2 928.71. During this time the dredge removed 42 900 cu. yds. of sand, at a cost of 6.8 cents per yard.

It is used not only to deepen channels, but also to fill in dikes and dams. This is quite an advantage where the amount of sediment carried by the stream is small.

This plant has been operated by Assistant Engineer J. Du Shane, and the above description is largely drawn from his report on the plant, which, together with other data, was obtained by the author from the U. S. Engineers located at Rock Island. The description here given applies to the dredge as remodeled in 1895.

This dredge evidently was regarded as fairly successful in that part of the river where first brought into use, as Major Ruffner, then in charge, says:*

"I believe that patience and some mechanical skill will make our machine able to dig a channel through any of our ordinary bars here, deep enough to induce the main flow of water through it in three days' work. * * * If this can be done there would be no difficulty in opening and maintaining a good navigable channel through this stretch of the river (between the Des Moines and Illinois Rivers) when the water is falling, by the use of two or three of these hydraulic dredges constantly operating and moving from the worst bar to the next one needing dredging."

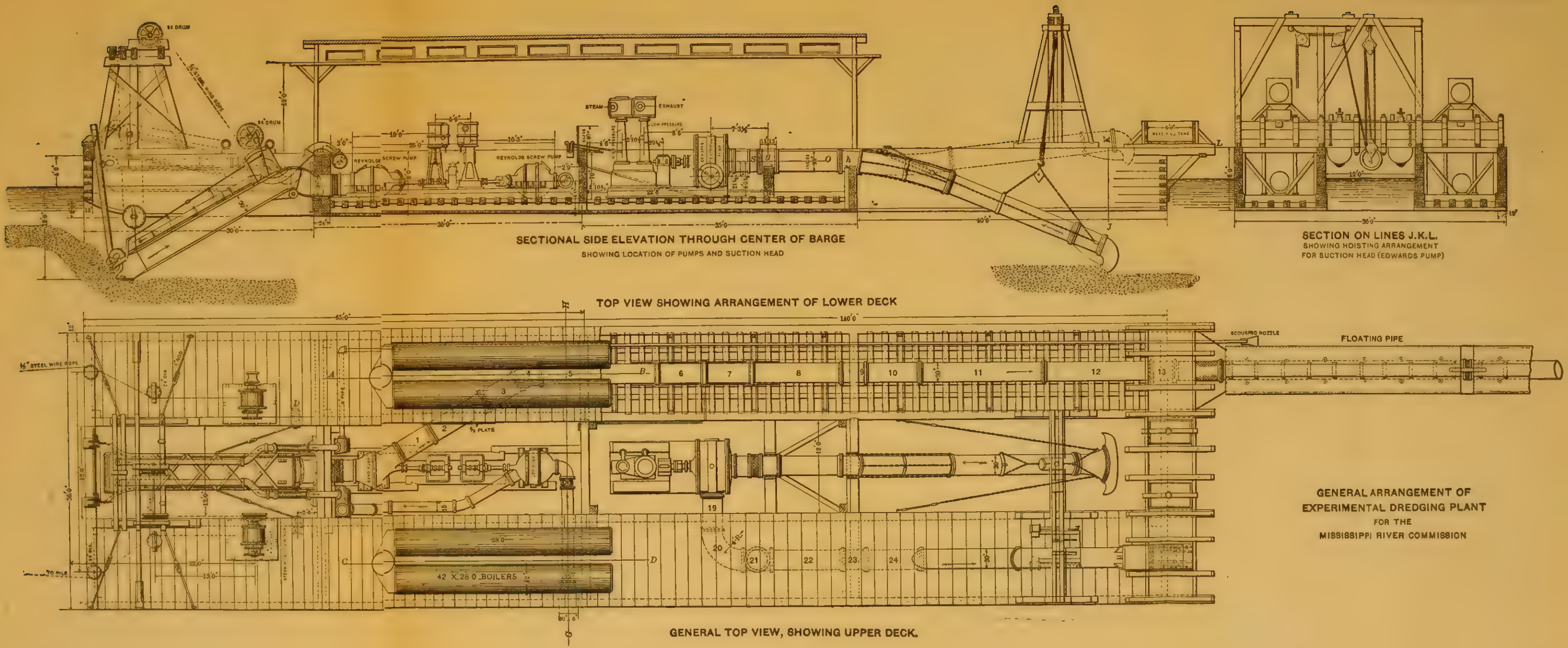
While this dredge was evidently a move in the right direction and was very useful in the upper river, its capacity was entirely inadequate to meet the requirements of the lower river. Here the dredging required at a single bar often exceeds 100 000 cu. yds., and the time permissible to remove this large amount of material is very short. To successfully accomplish this result machines were required of far greater capacity than any heretofore used or known.

In November, 1891, the representatives of steamboat transportation lines below St. Louis appeared before the Mississippi River Commission and urged immediate relief for low-water navigation. They stated that if such relief depended on the completion of the permanent improvement work, by the time it came, there would be no traffic. In response to this appeal, the Commission appointed a committee of two

"To study the subject of the construction and operation of such appliances for dredging as can be applied to the deepening of the

* Report Chief of Engineers, U. S. A., 1887, p. 1614.

PLATE XXXII.
PAPERS AM. SOC. C. E.
JUNE, 1898.
OCKERSON ON DREDGES AND DREDGING.



SECTIONAL SIDE ELEVATION THROUGH CENTER OF BARGE
SHOWING LOCATION OF PUMPS AND SUCTION HEAD

SECTION ON LINES J.K.L.
SHOWING HOISTING ARRANGEMENT
FOR SUCTION HEAD (EDWARDS PUMP)

TOP VIEW SHOWING ARRANGEMENT OF LOWER DECK

GENERAL TOP VIEW, SHOWING UPPER DECK.

GENERAL ARRANGEMENT OF
EXPERIMENTAL DREDGING PLANT
FOR THE
MISSISSIPPI RIVER COMMISSION

Mississippi River over the bars in extreme low water;" * * * and "Prepare and report a project for the construction of a dredging boat of as large capacity as can be handled in the channel of the Mississippi River at low water with safety and convenience."

Colonel Chas. R. Suter and Henry Flad, both Members of this Society, were designated to serve on this committee. After a thorough investigation of the matter, the committee concluded that among the various devices for temporary relief, dredging was the only means which held out a reasonable promise of success. The problem involved so many new features in the way of the enlargement and combination of devices heretofore used, that the construction of an experimental dredge was recommended. Drawings and specifications were prepared under the direction of this dredging committee, for an experimental dredge, and contracts for the construction of the various parts were let, at the close of 1892.

This experimental dredge was equipped with an Edwards centrifugal pump having a curved drag-suction at one end, and an Allis-Reynolds screw pump having a straight suction with jet agitators at the other end, the object being to make practical tests of the efficiency of the different devices. The pumps, engines and boilers were designed by their respective manufacturers. The Edwards pump runner was 6 ft. 4 ins. in diameter, and was operated by a compound, non-condensing, vertical engine with cylinders 15 ins. and 27 ins. in diameter, respectively, and with 20-in. stroke. The suction and discharge were each 30 ins. in diameter. The Allis-Reynolds pump was operated by a compound non-condensing vertical engine, with cylinders 8 and 16 ins. in diameter and with 12-in. stroke. The suction and discharge pipes were each 30 ins. in diameter. The jet pump was of the Allis-Reynolds pattern, with suction and discharge 15 ins. in diameter. This pump operated six jets 2½ ins. in diameter under a head of about 20 ft. By this means the material was loosened and divided up so as to enter the suction of the sand pump thoroughly mixed with water.

The steam for the various pumps, etc., was supplied from four boilers 38 ins. in diameter and 28 ft. long, of the type in common use on Mississippi River steamers. Hoisting apparatus was also

provided for raising and lowering the suction and moving the dredge.

The above-described machinery was mounted on a wooden hull 130 ft. long, 36 ft. wide and 8 ft. in depth. The dredge in working order drew about 4 ft. of water. A good general idea of this dredge can be obtained by inspecting Plate XXXII.

The material pumped was discharged through a floating pipe line made up of sections of pipe 33 ft. long, connected by means of strong rubber joints and iron coupling bars. An air chamber on each side of the discharge pipe served to float it when loaded with 10% or less of sand. In order to operate the dredge, two hydraulic piles are set about 25 ft. apart, near the upper end of the bar to be excavated. A wire rope about 1 000 ft. long is attached to each of these and to the hauling drums on the dredge. If the wind does not deflect the dredge from its proper place, it is allowed to swing freely in the current with the discharge pipes trailing below it. If necessary, side piles are set, and by means of lines attached to them, the dredge can be hauled into such position as may be desired. In dredging, the boat is pulled ahead up stream at rates varying with the depth of the material taken out, the average movement being about 60 to 75 ft. per hour. After one cut is finished, the dredge is dropped down again to the lower end of the bar, the head piles are moved over about the width of the cut and another cut is made near by and parallel to the first. This process is repeated until a channel of the required width and depth has been secured.

There is perhaps no single feature which facilitates this method of dredging as much as the hydraulic mooring piles (Fig. 7). The ease with which they can be set and withdrawn, and their great holding power, make them really indispensable where dredging is done in strong currents. They are sunk 15 or 20 ft. deep in the sand, and the head lines are attached to a shackle down at the surface of the sand, and consequently there is little danger of bending. A pile sinker provided with suitable leads and hoist, pumps and boilers, is required with each dredge. The pile is open at the lower end for the full size of the pipe. On the side of the pipe near the upper end is an opening 2½ ins. in diameter to which the pressure pipe from the pump is attached. The ease with which piles can be set and withdrawn is clearly set forth in Table No. 1, page 466.

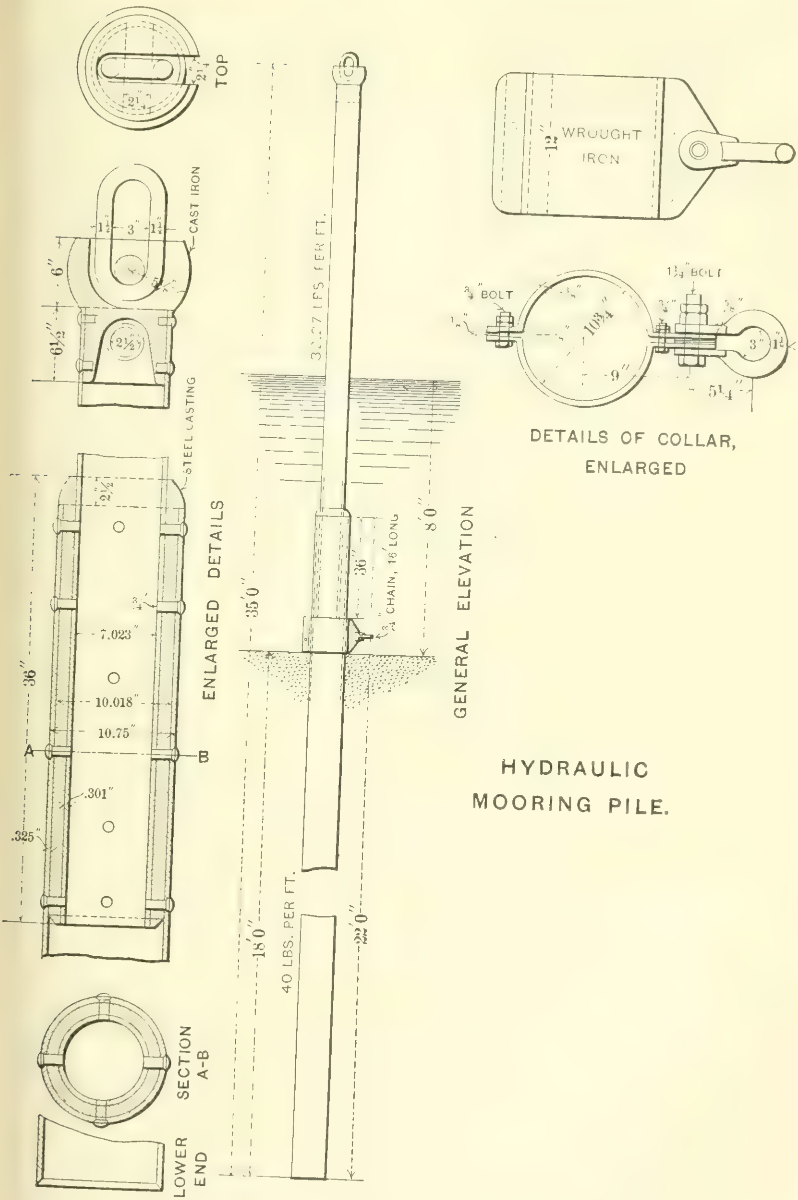

TOP
CAST IRON
STEEL CASTING
ENLARGED DETAILS
SECTION A-B
LOWER END
GENERAL ELEVATION
40 LBS. PER FT.
WROUGHT IRON
DETAILS OF COLLAR, ENLARGED
HYDRAULIC MOORING PILE.

FIG. 7.

TABLE No. 1.—EXPERIMENTS IN SINKING HOLLOW WROUGHT-IRON PILES IN SAND.

In these experiments one 6-in. inside-packed, double-plunger pump was used. The length of the piles was 34½ ft.; outside diameter, 11 ins.; thickness, ¾ in. for the heavy piles, which weighed 3 250 lbs. each. The light piles were of the same length and outside diameter; thickness, ½ in.; weight, 2 370 lbs. Diameter of water-pressure inlet 2½ ins. The piles were open at the bottom.

Number of test.	Depth of water.	Depth of penetration.	Pressure required.	Time required to sink.	Pile left undisturbed.	Time required to raise.	Boiler pressure.	Kind of pile.	Remarks.
	Feet.	Feet.	Lbs.	M. S.	H. M.	M. S.	Lbs.		
1..	6.0	19.5	40	3 46	1 05	1 12	70	Light	Both hoist and pump used to draw.
2..	7.5	19.5	40	3 34	1 15	1 35	65	Heavy	
3..	7.0	19.0	35	4 53	1 30	1 20	60	Heavy	Hoist only used to draw pile.
4..	8.0	20.0	35	6 00	1 50	1 35	73	Light	
5..	8.5	19.0	30	6 00	0 35	1 13	70	Heavy	Both hoist and pump used.
6..	8.0	18.0	30	4 29	16 00	1 25	68	Heavy	
7..	12.0	19.0	30	3 23	16 00	1 26	74	Heavy	
8..	12.0	18.0	35	3 26	1 55	1 11	68	Heavy	Hoist only used to draw.
9..	11.0	19.0	35	4 28	0 55	1 05	63	Heavy	
10..	2.5	20.0	55	4 00	0 22	1 58	80	Heavy	
11..	3.0	19.0	55	8 30	0 15	1 51	70	Heavy	

The various parts of the experimental dredge were assembled and erected under the immediate supervision of Henry Flad, M. Am. Soc. C. E., in the fall of 1893. The tests were begun in November of that year and continued throughout the winter and the following spring. As a result of these tests the Allis-Reynolds pump at the bow of the boat was found to be deficient in engine power and the drag-suction at the stern of the boat could not be manipulated so as to give a regular and satisfactory supply of material. The Edwards pump gave good results and the jet agitator suction-head was found to be satisfactory. The jet pump required extensive alterations before the required results could be secured.

The buoyancy of the floating pipe line was too small and great care was required to prevent sinking the pipes by getting a trifle over 10% of sand in them.

Another unexpected difficulty developed in the use of these pipes. When swinging freely in the current with the lower end open and the pump running, the pipes would kink up at the joints. This was attributed to the reaction of the discharge against the water in which

the pipes were floating. After considerable experimenting with jets discharging in the air and in water, the conclusion was reached that the kinking was due to the centrifugal force of the flowing water acting at the flexible joints of the pipe. That is to say, when the pipe swings freely in the current it is never straight, and any bend met with by the water flowing in the pipes at a high velocity is naturally exaggerated. This difficulty was fully overcome by attaching a baffle plate to the last section, a short distance from the end of the pipe, to receive the impact of the discharge. By shifting this baffle plate so that the discharge will strike it at an angle, the pipe line can be deflected to the right or left.

The experiments had solved many of the difficulties, and attention was then turned to utilizing this dredge for practical work. The Allis-Reynolds pump and the Edwards suction were discarded. The Edwards pump was shifted and attached to the jet suction. In the spring of 1895 a cabin was built for the accommodation of the crew and the first dredge for the elimination of obstructing sand bars on the lower river was ready for actual work during the low-water season of 1895.

In the fall of 1894 the first attempt to aid navigation was made near Cape Girardeau, Mo., where there was a bar about 1 600 ft. long with from 3 to 4 ft. of water. Work began on October 19th, and by October 26th a channel 6 ft. deep had been made and this channel remained good throughout the season.

During the winter of 1894–5 a large number of tests were made to determine the capacity of the dredge. The immense amount of material constantly moving along the bottom of the river and the scouring effect of the current, made it impracticable to ascertain the amount moved, by measuring the material in place and then measuring the excavation. A measuring barge was therefore fitted up with suitable valves, to attach to the lower end of the discharge pipe where the discharge could be deflected into the barge for a known interval of time when the running conditions were about normal. This barge is 107½ ft. long., 24 ft. wide and 6¼ ft. deep. A special floor was laid in the bottom, and tight bulkheads were built near each end. The gunwales were stiffened with trusses in order to carry with safety the unusual load put in the barge. A 6-in. gate-valve was placed below the floor, next to the bottom planking at each corner of the loading

space, or spoil bin, of the barge. Pipes lead along the gunwales to each of the valve pipes, and are arranged so as to be connected with the pressure pumps of the pile sinkers. A well with removable sides is placed around each gate-valve so that when the barge is filled the water may be drawn from the top without disturbing the sand. There are two gauges placed on each side of the spoil bin and near the ends of the same. By means of simultaneous readings on these gauges the depth of material is determined. In making a test the barge is attached to the lower end of the pipe-line by means of a rubber pipe 6 ft. long and of the same diameter as the floating discharge pipe. This carries the discharge into the barge and to a point near the forward bulkhead of the spoil bin. Here a valve is provided that will deliver the discharge either into the river through an opening in the bottom of the barge or into the spoil bin, as may be desired. This valve (shown in Fig. 8) operates, both in opening and closing, by releasing heavy weights which cause the valve to revolve almost instantaneously.

Fig. 8 shows the opening to the spoil bin closed and the outlet through the bottom of the barge into the river, open. The brakes shown hold the valve in any desired position. In setting the weights ready for a test, the weight *W*, which is shown at the bottom of the bin, is raised about 2 ft. above the button *O*, and supported by a rope attached to the framework above. It is also attached to the button *O* by a second rope. When ready to throw the valve down, so as to discharge into the spoil bin, the first rope is cut, which allows the weight *W* to drop about 2 ft., and the entire weight *W* is taken by the cable *K* which revolves the valve to its lower position, closing the outlet to the river and opening into the spoil bin. The brakes controlled by the lever *F* are put on to hold the valve in place, and the weight *W* is entirely released and drops to the bottom. The weight *A*, used for closing the valve to the spoil bin, is supported by the hook *N* some 2 ft. above the button *B* at the end of the cable *C* which passes through the weight, the opposite end passing around a pulley fixed to the valve shaft. When the spoil bin is full, the weight *A* is dropped from the hook *N* by raising the lever, and at the same instant the brakes are released and the valve moves back to its first position, throwing the discharge into the river. It is found necessary to start the valve with a jerk.

In making a test, the dredge pumps are started and the dredge is pulled ahead as in actual work, the barge-valve being set so as to discharge the spoil into the river. Sufficient water is let into the spoil bin through the gate-valves to cover the floor, and the gauges are read. This is done so as to be sure that none of the volume pumped in will go under the floor where it cannot be measured. When everything on the dredge is running smoothly and at about the normal rate, the dis-

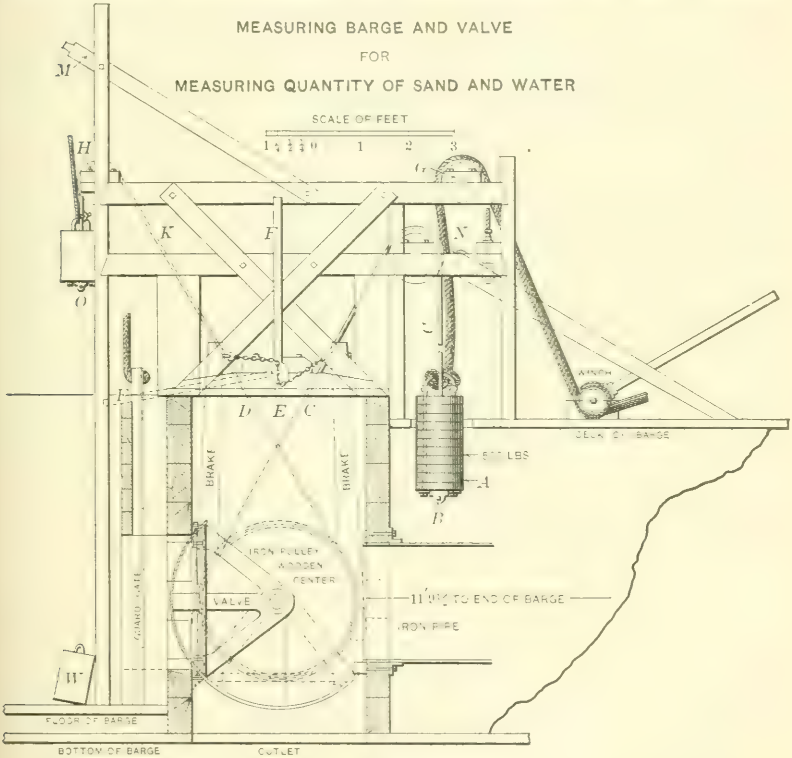


Fig. 8.

charge-valve into the spoil bin is thrown open and the timing hand of the stop-watch is started. In throwing the valve open to the spoil bin, the opening to the river is closed by the same valve. When all the material that the barge will safely hold has been discharged into the barge, the valve to the spoil bin is closed and opened to the river again. At the same instant the timing hand of the stop-watch is stopped. After a few minutes' time necessary for the agitated water in

TABLE No. 2.—CAPACITY AND EFFICIENCY TESTS

Number of test.	Duration of test.	Depth of Water at Bow. Port.	Depth of Water at Bow. Star.	Rate of movement ahead of dredge. Feet per second.	Edwards Pump. Revolutions per minute.	Edwards Pump. Steam pressure.	Edwards Pump. I. H. P. H. P. Cyl.	Edwards Pump. I. H. P. L. P. Cyl.	Edwards Pump. I. H. P. Total.	Edwards Pump. Head. Suction head. Mean head. Ins. of mercury.	Edwards Pump. Head. Suction head. Mean head. Feet of water.	Edwards Pump. Head. Mean delivery head. Feet of water.	Edwards Pump. Head. Mean velocity head by barge measurement. Feet of water.	Edwards Pump. Total mean head. Feet of water.	Jet Pump, 6 Jets, 2½ ins. Diameter. Delivery head. Feet of water.	Jet Pump, 6 Jets, 2½ ins. Diameter. Revolutions of jet pump.
	Secs.															
7......	150	6.5	7.3	.075	118	154	166	153	319	9.18	10.40	15.53	2.75	28.68	20.5	224
8......	138	7.0	8.0	.071	118	153	165	154	319	9.32	10.56	15.80	1.90	28.26	20.3	224
9......	122	5.5	6.5	.037	117	150	165	147	312	9.21	10.44	15.39	2.11	27.94	21.0	234
10......	140	6.0	6.5	.045	116	143	154	137	291	8.65	9.80	15.38	2.11	27.29	20.8	230
11......	155	6.5	7.3	.058	119	155	166	153	319	9.20	10.43	16.59	2.40	29.42	20.9	230
12......	138	6.5	7.5	.037	118	150	164	155	319	8.98	10.18	15.23	2.23	27.64	20.1	228
13......	123	8.2	8.3	.037	115	150	166	150	316	9.29	10.53	14.08	2.58	27.19	21.0	230
14......	135	7.0	7.7	.037	118	150	163	151	314	8.90	10.09	15.90	2.02	28.01	19.8	226
15......	131	8.2	8.0	.030	119	153	167	154	321	9.33	10.57	15.32	2.61	28.50	21.0	236
16......	183	10.0	9.5	.037	123	146	159	147	306	7.11	8.06	22.13	1.48	31.67	20.5	228
17......	194	5.5	5.0	.025	127	138	156	138	294	7.62	8.63	22.80	0.88	32.31	20.2	224
18......	197	5.5	8.0	.030	130	140	159	140	299	6.42	7.27	22.64	0.96	30.87	20.8	230
19......	174	5.0	5.0	.050	130	146	166	152	318	7.16	8.12	23.53	1.32	32.97	20.2	222
20......	193	7.0	6.2	.021	125	130	145	129	274	6.44	7.30	25.52	1 24	34.06	19.8	216
21......	184	3.5	4.0	050	129	143	167	151	318	7.17	8.13	23.36	1.39	32.88	21.0	230
22......	180	5.0	5.0	.050	120	140	155	145	300	7.68	8.70	20 73	1.80	31.23	20.0	226

The first nine tests were made with 604.5 ft. of pipe between the pump and the valve
Diameter of discharge pipe, 30 ins. Distance from suction head to pump, 101 ft.
The area of spoil bin in test barge, 2 544.01 sq. ft.
The average capacity of sand per hour with the short discharge pipe was 679 cu. yds.,
The sand pumped per hour per indicated horse-power with the short discharge was

the spoil bin to settle down, the gauges are read, the pumps on the pile sinkers are started, the gate-valves are opened and the water is forced out, leaving the sand to be measured later. This is done by measuring the depth on eleven cross-sections at eleven points on each section, all equally spaced. The average depth multiplied by the known area gives the volume of sand. The differences of the gauge readings before and after filling give the depth of material pumped in. This multiplied by the area gives the total volume of sand and water. From this, and the time required to pump such volume, the various functions of capacity per hour, velocity per second, efficiency, etc., are determined. The material enters the barge with such force that the sand is not distributed uniformly over the bottom, but piles up in ridges on one side or the other.

OF UNITED STATES DREDGE *ALPHA*. 1894–95.

Measuring Barge.									Sand.		Efficiency.		
Mean depth of water in barge before test.	Mean depth of water in barge at end of test.	Mean depth of water pumped.	Cubic feet of material pumped.	Cubic feet per second.	Velocity per second. Feet.	Cubic feet of sand pumped.	Percentage of sand.	Capacity of pump, cubic yards.	Weight of 1 cu. ft., dry.	Percentage of voids.	Work done in foot-pounds per second as measured by barge.	Work done in foot-pounds per second as measured by indicator cards.	Percentage.
								Per Hour					
.626	4.478	3.852	9 799.52	65.33	13.31	552.0	5.63	491	106	35	117 106	175 450	66.75
.707	3.663	2.956	7 520.09	54.49	11.10	936.5	12.45	905	104	33⅓	96 244	175 450	54.85
.575	3.340	2.765	7 034.18	57.66	11.74	766.5	10.89	838	104	38	100 688	171 600	58.67
.438	3.610	3.172	8 069.60	57.64	11.74	614.0	7.61	585	108	36⅔	98 312	160 050	61.42
.406	4.120	3.714	9 448.45	60.96	12.41	970.0	10.27	835	104	37⅓	112 062	175 450	63.87
.505	3.713	3.208	8 161.18	59 14	12.04	520.0	6.37	503	109	36⅔	102 187	175 450	58.24
.398	3.450	3.052	7 764.31	63.12	12.85	715.0	9.21	775	109	33⅓	107 250	173 800	61.71
.473	3.465	2 992	7 611.67	56 38	11.48	660.0	8.67	652	106	36⅔	98 687	172 700	57.14
.448	3.720	3.272	8 324.00	63.54	12.94	516 3	6.20	526	106	38⅓	113 187	176 550	64.11
.562	4.018	3.456	8 792.09	48.04	9.78	460.0	5.23	335	100	41⅔	95 062	168 300	56.48
.438	3.260	2.822	7 179.19	37.01	7.53	530.0	7.38	364	108	38⅔	74 750	161 700	46.23
.461	3.448	2.987	7 598.95	38.57	7.85	925.0	12.17	626	106	36⅔	74 437	164 450	45.27
.557	3.647	3.090	7 860.99	45.18	9.20	716.0	9.11	549	108	37⅓	93 125	174 900	53.25
.530	3.857	3.327	8 463.92	43.85	8.94	586.6	6.93	405	111	34⅓	93 375	150 700	62.62
.616	3.975	3.359	8 545.32	46.44	9.46	663.0	7.75	446	107	40	95 438	174 900	54.57
.643	4.392	3.749	9 537.49	52.98	10.79	849.0	8.90	629	103	41⅔	103 438	165 000	62.69

in the test barge. The other seven tests were made with 1 059.5 ft. of pipe.
Diameter of suction pipe, 30 ins. The suction was lowered 12 ft. below the water surface

and with the long pipe, 479 cu. yds.
2.16 cu. yds., and with the long line, 1.59 cu. yds.

Sometimes the sand is measured in a box 1 ft. deep, 5 ft. wide, and 10 ft. long. The floor of the barge is cleared off so as to give room for the box. The sand is then shoveled into it without packing, and the top is smoothed off with a straight edge. This method gives about 22% more volume than when the sand is measured in place, but as it necessitates handling the material twice, it has been abandoned.

The results of twenty-two tests of the dredge *Alpha* with the Edwards pump are given in Table No. 2.*

The time spent in experimenting with the first dredge was by no means wasted. There were many difficult problems of construction and manipulation solved, and the efficacy of dredging at all in a stream which moves vast quantities of material on its own account

* Report of C. W. Sturtevant in the report of Chief of Engineers, U. S. A., 1895, p. 3795.

was tested to some extent. The results of all these experiments pointed clearly to dredging as the most promising method of temporarily improving low-water navigation that had so far been suggested or tried. The time had now come to provide a sufficient number of dredges to clear the obstructing reefs in so short a period of time that the interruption to navigation would be very slight. Dredging, however, had not yet been authorized specifically by Congress, and the amount of money available for the construction of dredges was rather limited.

With a view to covering the ground still more fully, and to get the benefit of the best dredging experience available, a circular was sent out in August, 1894, to engineers, contractors and builders of dredges, inviting "Informal plans and suggestions as to the methods and machinery best adapted to the work." The capacity of the required dredge was to be 1 600 cu. yds. of sand per hour. Several replies were received, and three firms were requested to prepare detailed drawings and specifications of the plant they proposed to furnish. These were submitted in December, 1894, and a contract was let to the American Hydraulic Dredging Co. of Chicago, Ill., for the construction of the dredge *Beta*, at a cost of $172 775. The contract provided that if the dredge fell short of the required capacity, the deduction from the price should be proportional to this difference; and if the capacity exceeded 1 600 cu. yds. per hour, then the price should be increased proportional to this excess up to 50 per cent. The dredge was completed and ready for testing in January, 1896.

Dredge *Beta*.

This dredge has two independent dredging machines complete from suction to end of discharge pipes. The sand pumps are of the centrifugal pattern with runners 7 ft. in diameter and with 8 arms in each runner. The suction for each pump is 33¾ ins. in diameter, and is split near the pump so that the water enters the casing from both sides. The discharge for each pump is from a single 33-in. pipe leaving the pump chamber from the top. The suction for each pump divides near the forward end of the hull into three suctions, each 19½ ins. in diameter. A cast-iron elbow is riveted to the hull at the bow for each of the three branches, which are provided with elbows of hammered copper ⅜ in. thick, and these work in the iron elbows and form

a radial, telescopic joint. The axis of rotation is the center of a 7-in. shaft supported on brackets at the bow of the boat. The elbows are concentric with this shaft. These three suctions are framed together and can be raised or lowered, as one piece, through a depth of 20 ft.

Each of these suctions is provided with a vertical revolving cutter, 5 ft. in diameter and 5 ft. long, with 12 nickel-steel blades on each cutter. These serve to loosen up the material so that it will readily enter the suction. They are spaced 6 ft. apart from center to center. There are six of these revolving cutters for the two pumps. They are driven, through a system of spur gearing, by a cross-compound, non-condensing engine with 14½ and 29-in. cylinders and an 18-in. stroke. The engines make about eight revolutions to one revolution of the cutters. The cylinders are some 20 ft. apart and are connected by a 5-in. pipe.

To support a portion of the weight of the cutter machinery, a pontoon is provided for each set of suctions. This pontoon is made of steel plates, and is built around the various pipes and frames. Each pontoon has a displacement of 1 000 cu. ft.

The sand pump shaft is of forged steel, 10 ins. in diameter, and 12 ins. in diameter where it passes through the pump runner. Stuffing-boxes are provided where the shaft passes through the pump casing. Each pump is operated by a direct-connected, triple-expansion, vertical, inverted, four-cylinder, tandem engine provided with jet condensers. The cylinders are 20½, 33, 38 and 38 ins. in diameter, respectively, and the stroke is 24 ins. with the cranks set at right angles. These engines are run at a speed of about 130 revolutions per minute, the indicated horse-power of each being about 1 250, with boiler pressure of 175 lbs. Each engine is provided with a Worthington duplex air pump and jet condenser, with cylinders 14 and 19 ins. and 15-in. stroke. They are also arranged to exhaust without condensing. The steam is supplied by four Heine boilers so arranged that they can be used together or independently, as may be desired. Each boiler has two shells 36 ins. in diameter and 18½ ft. long. The water legs are connected by 171 tubes, 3½ ins. in diameter and 15¾ ft. long. There is one smokestack to each pair of boilers, each stack being 64 ins. in diameter and 57 ft. high above the breeching, or 73 ft. above the water-line. The main steam pipe between the boilers

and engines is 12 ins. in diameter. The usual steam gauges, blow-off valves, etc., are provided.

The water supply for the boilers and journal bearings is filtered by means of two filters, each 11 ft. long and 6 ft. in diameter. The filter bed is supported on a perforated cast-iron bed fixed near the bottom of each filter. There are three layers of brass wire-cloth with 10, 60 and 10 meshes per inch, respectively. These support a bed of crushed quartz made up of layers of different degrees of fineness and about 2 ft. thick. The sediment which gathers on the filter bed is stirred up by two sets of arms which can be revolved by means of gearing, and the clear water from one filter is used to wash the other. The filters are supplied by a duplex pump with 8-in. suction and 7-in. discharge, and the water is carried through an alum coagulant to the tops of the filters under a pressure of about 60 lbs. The filtered water is drawn from the bottom of the filters by the boiler feed-pumps, and passes through a vertical pressure heater provided with 100 2-in. corrugated copper tubes $73\frac{3}{4}$ ins. long. The exhaust steam from the cutter, the winding engines and the pumps passes through this heater.

The winding engines are placed near the bow of the boat. There are six drums operated by a double-cylinder, link-motion, reversing engine with $12\frac{1}{4}$-in. cylinders and 15-in. stroke. Four of the drums are 24 ins. in diameter and 40 ins. long. Two of these drums are used for side warping, and are geared 25 to 1. The other two are used for pulling the dredge ahead, and are geared 30 to 1; 1 200 ft. of $\frac{7}{8}$-in. wire rope is used for this purpose. The two end drums are 30 ins. in diameter and 28 ins. long. They are used for raising and lowering the suctions.

At the stern of the boat is a steam capstan and a spud hoist. There are two vertical anchor-spuds and one push-spud at the stern of the boat. The anchor-spuds are of oak, 24 ins. square and 40 ft. long. These were intended mainly as pivots on which to swing the boat, so that a wide cut could be made by moving the suction on the arc of a circle. This method is not well adapted to streams with a strong current. These spuds and the push-spud are raised or lowered by means of a 3-drum friction hoist moved by a double 8 x 10-in. engine. The push-spud is an iron beam 28 ft. long, with a steel shoe at the lower end. It can be raised up into a recess in the hull

when not in use. The inner end of the beam is attached to the piston rod of a hydraulic cylinder 7 ft. long and 15 ins. in diameter. The piston rod passes through the hull plating in a stuffing-box. This spud has never been used.

The machinery is mounted on a steel hull 172 ft. long and 40 ft. wide. It is 7 ft. 2 ins. deep in the waist, and for 36 ft. from the forward end and 16 ft. from the after end it is 10 ft. 10 ins. deep. These deep portions are decked over, as are also the guards, for the entire length. The plating on the sides of the hull is $\frac{3}{8}$ in. thick; the other plating outside is $\frac{5}{16}$ in. thick. The bow is double-plated on the head and rake. The cross floors, spaced 2 ft. between centers, are 12-in. I-beams, weighing 32 lbs. to the foot. On these, at intervals of $3\frac{1}{2}$ ft., are longitudinal stringers of the same form and height, weighing 40 lbs. to the foot. These floor frames are decked over with $\frac{1}{4}$-in. plating and extend to within 6 ft. of the bow. The space between this plating and the bottom is divided into ten water-tight compartments. The hull is further divided into seven water-tight compartments above the water bottom by means of five cross bulkheads and one short longitudinal bulkhead.

The bases of the pumps, engines and the boilers are set on the water bottom. Two coal bunkers are provided, with a capacity of about 30 tons each. The dredge is fitted with a traveling crane of ample capacity to handle all parts of the machinery.

No quarters are provided for the crew of this dredge. The cabin is only sufficient to protect the machinery. The boat is equipped with an electric light plant and a full complement of shop tools, such as lathe, drill press, etc.

The requirements of the contract fixed the limit of draft at $4\frac{1}{2}$ ft., and the width at 40 ft. When the dredge was completed, the draft was found to be about $6\frac{1}{2}$ ft.

The weights of the various parts are about as follows: Hull, 633 340 lbs.; main engines and pumps, 183 576 lbs.; boilers, 405 880 lbs.; cutter engine, 27 460 lbs.; forward hoist, 51 500 lbs; spud hoist, 20 450 lbs.; ladders, 56 800 lbs. The total displacement when running, with suction and discharge pipes, boilers, filters, heaters, etc., full of water and a supply of coal on board, is about 1 175 tons.

The discharge consists of two independent pipe lines, each 1 000 ft. long. A deflecting baffle plate is provided for the end of each line.

TABLE No. 3.—Capacity and Efficiency

Number of test.	Duration of tests.	Depth of water.	Depth of suction.	Forward movement per minute.	Pumps' Revolutions per minute. Starboard.	Port.	Average boiler pressure.	I. H.-P. Starboard engine.	Port engine.	Total.	Suction head. In feet of water. Starboard pump.	Port pump.	Delivery head. In feet of water. Starboard pump.	Port pump.	Difference between elevation of gauges.	Velocity head.	Total head. Starboard pump.	Port pump.
	Secs.	Ft.	Ft.	Ft.											Ft.			
1	35	8.3	14.6	7.5	150	149	177	1 280	......	2 561	19.3	19.8	22.7	25.2	+8.4	6.4	57.1	60.1
2	685	11.5	18.5	3.1	140	157	169	.....	1 296	2 593	15.9	18.2	30.6	34.5	.....	2.3	57.1	63.3
3	37	10.0	18.5	2.6	161	157	161	1 293	......	2 586	14.7	14.7	36.3	39.7	.....	2.1	61.5	62.9
4	647	9.8	18.3	3.6	151	148	178	.....	1 223	2 447	14.7	17.0	35.1	26.1	.....	2.9	61.1	54.4
5	261	9.2	17.5	5.0	153	154	176	1 186	......	2 392	20.4	18.7	29.4	37.4	.....	2.8	61.0	67.3
6	553¾	9.5	17.6	4.0	144	151	173	.....	1 300	2 609	18.7	18.1	25.5	32.8	.....	3.0	55.6	62.3
7	53	9.5	17.5	7.5	146	148	172	1 105	......	2 211	17.0	19.0	31.7	27.6	.....	3.3	60.4	58.0
8	50	10.1	18.4	10.0	147	148	174	.....	1 247	2 495	20.0	21.5	28.3	29.2	.....	2.8	59.9	61.9
9	1023¾	9.5	18.7	8.6	141	143	176	1 170	......	2 340	19.3	21.0	23.8	24.9	.....	2.5	54.5	57.3
10	95	9.5	18.8	8.2	142	144	174	.....	1 199	2 398	19.3	20.4	24.9	24.7	.....	3.7	56.3	57.2
	119	.....	.....	.....	149	152	170	1 187	......	2 374	18.1	18.9	21.5	21.5	.....	4.0	52.1	52.8
	78 1/10	4.5	11.5	3.7	146	157	163	.....	1 161	2 322	17.6	13.6	22.7	35.0	.....	3.2	51.9	60.2
	......	.....	.....	.....	.....	.....	.....	......	......	......	.....	.....	.....	.....	.....	.....	.....	.....

Length of port suction from intake of pump = 85.9 ft. Length of port discharge pipe
Length of starboard suction from intake of pump = 100.3 ft. Length of starboard
Diameter of main suction = 33¾ ins. Diameter of discharge pipes = 33 ins. Area of
Revolutions of cutters per minute = 19. Average capacity of dredge in cubic yards

Eight of the sections of each line are 100 ft. long, and four sections are 50 ft. long. When the pipes are empty, the pontoons draw about 14 ins. of water. The pontoons are made of $\frac{3}{16}$-in. tank steel, and are fitted with water-tight bulkheads about 16 ft. apart. In section they are the shape of a semi-cylinder, with an outer radius of 30¾ ins., and an inner radius of 16¼ ins. The discharge pipe lies in the float thus formed, and is made of ¼-in. steel. The different lengths are coupled about as described in connection with the *Alpha*.

Quite a number of defects were developed in the use of this dredge, many of which were remedied. At first the cutters all worked in the same direction, and the result was that the dredge was pulled sidewise out of the cut. This was remedied by changing one set of cutters to work in the opposite direction. The breaks in the gearing of the cutter machinery were frequent, although the spur gears were made of

TESTS OF UNITED STATES DREDGE *BETA*.

Number of test.	Mean velocity. Feet per second.	Total material pumped per minute.	Amount of sand pumped per minute.	Percentage of sand.	Quantity of sand pumped per hour.	Depth of material pumped into barge.	Percentage of voids in wet sand.	Weight per cubic foot of dry sand.	Horse power developed as measured by material in barge.	Efficiency per centum.	Sand pumped per hour per indicated horse power.	REMARKS.
	MEASURING BARGE.											
	Ft.	Cu. ft.	Cu. ft.		Cu. yds.	Ft.		Lbs.			Cu. yds.	
1	20.8	14 812	391	2.6	868	3.41	35.2	96.9	1 685	65.8	0.29	Pumping sand and water.
2	11.9	8 479	1 281	15.1	2 847	3.82	32.6	98.1	1 608	42.3	1.10	
3	11.6	8 268	2 859	34.6	6 354	2.63	34.2	97.5	1 298	50.3	2.46	
4	13.8	9 809	2 245	22.8	4 988	4.18	33.7	98.8	1 298	53.0	2.04	
5	13.5	9 608	3 509	36.4	7 798	1.65	34.8	98.1	1 558	65.2	3.26	
6	14.0	9 956	2 183	21.9	4 850	3.77	35.1	97.8	1 335	51.2	1.86	
7	14.0	10 000	1 743	17.4	3 872	3.54	32.5	99.4	1 300	58.8	1.75	
8	13.5	9 635	2 798	29.0	6 217	3.20	33.3	97.5	1 399	56.1	2.49	
9	13.9	9 870	2 847	28.9	6 327	3.33	34.6	97.5	1 318	55.3	2.70	One valve open.
10	14.9	10 639	2 287	21.5	5 082	3.35	34.8	97.5	1 367	57.0	2.12	One valve open.
	16.5	11 430	....	....	....	4.51	....	......	1 134	47.7	....	Water test.
	14.4	10 505	499	4.7	....	1.83	....	......	1 165	50.2	0.48	Gravel test.
	14.1	10 107	2 214	23.0	4 920	3.22	....	......	1 365	55.5	2.00	Averages of first ten tests.

from pump to valve in measuring barge = 1 161.7 ft.
discharge pipe from pump to valve in measuring barge = 1 147.7 ft.
measuring barge = 2 535.3 sq. ft.
per hour by box measurements = 4 920. or about 4 330 cu. yds. measured in place.

nickel steel, and the pinions of phosphor bronze. The 6-drum hoist also gave considerable trouble. The cutters were found to be very much longer than necessary, and their operation drew heavily on the boiler capacity. The unexpected draft developed difficulties in coaling, and in maneuvering the dredge over the shoal bars.

The preliminary tests of the dredging machinery began about the middle of January, 1896, and continued until March 1st. Of the four hundred working hours embraced in this period, two hundred and fifty-five hours were used in repairs and changes and seventy-four hours in actual pumping. The capacity tests were begun on March 10th, and continued for about a month. There were ten tests made in ordinary river sand, one gravel test and one water test. The tests were made in the test barge described in the preceding pages. Two valves were provided, so that both pipe lines discharged into the barge.

In the gravel test the face of the bank dredged was about 7 ft. high, and only about 1 ft. of it was gravel. In this test one valve was first opened and the barge filled. The water was then syphoned out, and the other valve was opened and the barge again filled. The gauge readings and time of each were, of course, noted. Tests 9 and 10 were made with only one valve open, the other pipe line discharging in the river.

In all the tests the dredge was run for a considerable time in all respects as if doing actual dredging work, and the valves were opened at such times as the pumps were seen to be working under about normal conditions. In the mean time steam gauges were read, revolutions of pumps, cutter engines and winding drums were counted, and indicator cards were taken at intervals of about 5 minutes.

The results of these tests are shown in Table No. 3.*

The general conditions as to length of time required for efficiency tests and number of capacity tests were the same as described in connection with the *Gamma.*

The tests show an average capacity of 4 920 cu. yds. per hour, and, therefore, according to the terms of the contract, a bonus of $86 387.50 was earned by the contractor.

After completing the required tests, sundry repairs and alterations were made, to put the dredge in condition for actual work.

During the low-water seasons of 1896 and 1897 the enormous capacity of the *Beta* was fully demonstrated. An account of the character and cost of this work will be given later. Before the dredge is again put into service for the low-water season of 1898, numerous changes will be made. The hull will be widened to 56 ft. and lengthened, so as to reduce the draft to 4½ ft. About $41 000 has been retained from the contract price to do this work, as the specifications required that the draft should not exceed 4½ ft. A cabin for the accommodation of the crew will be erected; the 6-drum hoist will be replaced by four detached single-drum hoists; the cutter engines, cutters and other parts belonging to the present suction will be replaced by a jet suction and pumps. This is done because it has been found in actual practice that mechanical agitators are unnecessary in the sands of the Mississippi River. The wear and tear of the cutter machinery is a very large item of expense, to say nothing of the power

* Report of Assistant Engineer Wm. Gerig, in report of Chief of Engineers, U. S. A., for 1896, page 3643.

FIG. 1.

FIG. 2.

required. The jet agitators are quite efficient, are much more economical in point of repairs and power required, and are more simple in manipulation. The push-spud and one anchor-spud will be discarded. The other anchor-spud will be moved to the bow of the boat. The spud-hoist will be discarded. The forward battery of boilers will be turned end for end in order to properly distribute the weights under the new arrangements, and to improve the facilities for coaling. Much of this work would probably be deferred were it not that the boat must be put into dry dock to make the hull repairs, and this opportunity is seized for making the changes outlined.

A general view of the *Beta*, as she will appear after remodeling, is shown in Fig. 9.

Dredge *Gamma*.

The Act of Congress of June 3d, 1896, was the first to formally recognize and require the use of dredge boats and other devices as an adjunct to the permanent improvement of the lower Mississippi River. The Act referred to requires that as much of the money appropriated as may be necessary

"Shall be expended in the construction of suitable dredge boats and other devices and appliances, and in the maintenance and operation of the same, with a view to ultimately obtaining and maintaining a navigable channel from Cairo down, not less than 250 ft. in width and 9 ft. in depth, at all periods of the year, except when navigation is closed by ice."

This formal recognition of the necessity of temporary relief for low-water navigation resulted in the early construction of four more dredges.

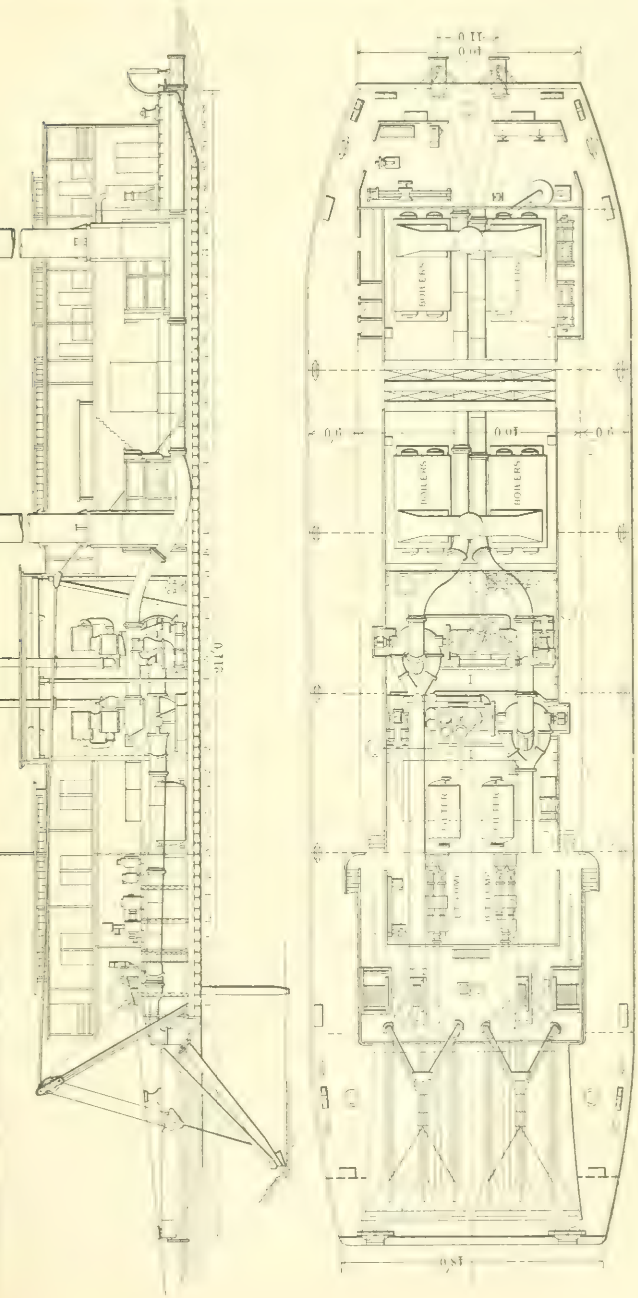
The contract for the *Gamma* was let in July, 1896, to the Bucyrus Steam Shovel and Dredge Co., at South Milwaukee, Wis. The capacity required was 800 cu. yds. of ordinary river sand per hour, delivered through 1 000 ft. of pipe with a single centrifugal pump. The price paid was $85 530.60, and the time fixed for completion was nine months. This dredge was used in actual work throughout the low-water season of 1897, and was found to be very satisfactory, both in capacity and economy.

The hull of the *Gamma* is of steel, and is 138 ft. long over all, 38 ft. wide and 8 ft. deep. There is a well at the bow 32½ ft. long, in which the suction is placed.

The framing of the hull consists of 12-in. channels, stiffened with I-beam keelsons. The framing is shown in Fig. 1, Plate XXXIII. Cross bulkheads divide the hull into two compartments, one for the boiler, and the other for the engines. These are surrounded by water-tight compartments and are decked over.

The main dredging pump is located in the forward part of the engine room. It is a centrifugal pump, with a 24-in. suction inlet on each side and a 34-in. discharge at the bottom. The runner is 5 ft. 9 ins. in diameter, and has four blades. It revolves at a speed of about 150 revolutions per minute. The pump casing is split horizontally through the axis, so that the upper half can be removed in making repairs. The casing is of cast iron, from $1\frac{3}{4}$ to $2\frac{1}{2}$ ins. thick. The shaft passes through the pump, and has a bearing on each side of the casing. At these bearings the shaft is 8 ins. in diameter. The pump is driven by a cross-compound, condensing engine with an independent condenser. The high and low-pressure engines are horizontal, and are connected to two disc cranks set at right angles and keyed to opposite ends of the pump shaft. The high-pressure cylinder is 18 ins. and the low-pressure $32\frac{1}{2}$ ins. in diameter; the stroke is 22 ins. The high-pressure cylinder has an adjustable cut-off, with a hand adjustment to regulate the point of cut-off. Expansion relief-valves are provided for the low-pressure cylinder. These engines develop about 500 H.-P., with a boiler pressure of 140 lbs. The jet pump is of the centrifugal type, with 18-in. suction and discharge. This pump supplies the water to stir up the sand at the suction head. The runner has four arms, with steel tips, running as close as practicable to the casing, to obtain the greatest available pressure. This pump and its engine rest on a common bed plate, and have a flange-coupled shaft for pump and engine. The engine is a compound condensing engine of the marine type. The high-pressure cylinder is 12 ins. and the low-pressure 22 ins. in diameter, with a common stroke of 14 ins. The crank shaft has the cranks set at right angles. The cylinders have plain slide valves. All parts are provided with automatic apparatus for lubrication.

The air pump is horizontal, with single steam cylinder 10 ins. in diameter, air cylinder 18 ins. in diameter, and 18-in. stroke. The condenser receives the steam from both main and jet pumps, and is so arranged that the steam can pass directly to the atmosphere. The overflow from the condenser passes through a chamber, forming a hot-



HYDRAULIC DREDGE BETA

PLAN AND ELEVATION SHOWING ALTERATIONS REQUIRED TO PROVIDE CABIN ACCOMODATIONS AND REDUCE DRAFT.

FIG. 9.

well for the boiler feed pumps. There are two feed pumps and one fire pump, so arranged that any one of them can be connected with the feed-water pipe or the fire hose. These pumps are vertical, duplex, outside-packed plunger pumps, with 7½-in. steam cylinders, 4½-in. water plungers and 8-in. stroke. An ordinary sand filter, with alum as a coagulent, is provided to filter the water used in the galley and cabin. A separate pump forces the water through the filter to a tank on the roof, and from there it is piped to various parts of the boat.

The main hauling winches are located on opposite sides of the boat near the bow. They are driven independently and are provided with fast and slow feed controlled by clutches. The drums are 42 ins. in diameter and 42 ins. long, which gives space for 1 200 ft. of ¾-in. wire rope. Each drum is driven by a pair of 7 x 7-in. double cylinder engines with cam reverse, and the power is transmitted through a combination of worm and spur gearing. The movement is controlled by means of a brakewheel and strap. The speed ratio is 81 to 1 for fast and 493 to 1 for slow speeds. The hoisting engine for raising the suction pipe has a drum 24 ins. in diameter and 24 ins. long. It is fitted with brake, spur gear, pinion, clutch and shifting device. This engine also operates a shaft terminating in gipsy heads, which can be used in lieu of the capstans. The whole is driven by a pair of horizontal, double-cylinder, 8 by 10-in., plain, slide-valve engines, provided with steam reverse.

The anchor spud is hoisted by means of an upright, single-acting, steam cylinder, 18 ins. in diameter and 6-ft. stroke. The piston is attached to a cam clamp at its upper end, which grips and holds the spud against the slides. Steam enters at the bottom of the cylinder through a three-way valve, raising the piston and spud to the height of its movement. When the valve is turned to the exhaust port the piston and clamp descend, the spud being supported by a rack and pinion. The pawl locking the pinion is hinged between its center of movement and point, and the tripping is effected by making the pawl buckle on itself so that the point turns under. This releases its hold and allows the spud to drop.

Steam for the above-described plant is furnished by six boilers 48 ins. in diameter and 28 ft. long, with five 11-in. flues in each. These boilers are arranged in two batteries so that they can be used separ-

ately if desired. They are designed to carry a pressure of 140 lbs. The fire-bed and ash-pan rest directly on the keelsons. Directly in front of the fire doors are coal bunkers having a capacity of 40 tons. The bottoms of the mud drums are below the water-line, hence a special cleaning device is provided by means of a tank set between the keelsons under the mud valves. The mud and water remaining in the drums are drawn off into the tank, and then syphoned out. The feed water is heated by passing through two pipes hung in brackets from the sides of the parting wall between the two battery furnaces.

The two intake pipes at the pump casing run forward to the suction head. They are hinged to the boat at the bow by two flanges connected by a pivot pin on the lower side and provided with a circular stuffing-box, making a radial joint that will allow the suction to be lowered to a depth of 15 ft. Outside of the joint the pipes and suction head are framed together and rigidly connected so as to form essentially one piece. At the suction head the pipes are 11 ft. apart from center to center. The suction head for each pipe is 8 ft. long and they are 3 ft. apart, forming one suction head 19 ft. long over all. The suction head proper has two inlets, one for the upper and one for the bottom intake, and between the two is a triangular pressure chamber running the whole length of the head. This connects directly with the jet pump. This chamber has nine $2\frac{1}{4}$-in. nozzles through which the water from the jet pump is forced with sufficient pressure to loosen the sand in front of the suction. The discharge pipe is 34 ins. in diameter. It runs from the bottom of the pump casing along the floors between the two central keelsons, under the partition walls between the batteries of boilers, and through the stern of the hull, with the center of the pipe 4 ft. above the bottom of the hull at its exit. Near the pump the entire pipe is below the water-line, and the priming is done by means of a steam syphon at the top of the pump casing. The floating discharge pipe is 1 000 ft. long, divided into 20 sections, each 50 ft. long. They are made of $\frac{1}{4}$-in. tank steel. The pipes are floated by means of 40 cylindrical pontoons, each 23 ft. 9 in. long and 30 ins. in diameter. These are made of $\frac{3}{16}$-in. tank steel. They are attached to the discharge pipe by means of truss frames constructed of bar and angle irons, so designed as to receive and clamp the discharge pipe and carry the pontoons on either side. The pipe line is attached to the hull section of the discharge pipe by means of a male

and female bevel-flanged coupling, a davit being used to swing the floating pipe into position.

The hull has a cabin 97 ft. long and 29 ft. wide, which protects the machinery and provides quarters for the crew. There is also a repair shop, provided with a lathe, drill press, emery grinder and other appliances needed in making ordinary repairs. An electric plant furnishes light for one 4 000-candle-power search-light, four arc-lights of 1 200-candle-power each, and 75 incandescent lamps of 16-candle-power each.

All the winches, the spud hoist and the search-light are manipulated from the operating room. This dredge is towed from point to point, and while dredging is operated similarly to the *Alpha*.

Four 10-in. and four 12-in. hydraulic piles 33 ft. long are provided, similar to that shown in Fig. 7. A general elevation of this dredge is shown in Fig. 2, Plate XXXIII, and the arrangement of the machinery, etc., may be seen in Fig. 10.

This dredge was tested near St. Louis, Mo., in August, 1897, and, after the completion of the tests, she was used in actual dredging between Cairo and Memphis until early in December of the same year. The general efficiency test, called for by the contract, required that the dredge should be operated 60 working days of 12 hours each in water from 5 to 15 ft. deep, and with sand of such different degrees of coarseness as will be found on the low-water bars. After this had been done and the machinery found satisfactory, then 20 capacity tests were required to be made with the suction at different depths. This last requirement was considered filled when the total amount pumped per hour divided by 20 was equal to or exceeded the required capacity of 800 cu. yds. per hour.

The weights of the principal parts are as follows: Hull, 328 328 lbs.; cabin, 117 449 lbs.; main pump and engines, 74 722 lbs.; jet pump and engines, 21 717 lbs.; auxiliary engines, drums, levers, etc., 46 937 lbs.; air, feed and fire pumps, 15 695 lbs.; electric light plant, 7 700 lbs.; boilers and accessories, 325 911 lbs.; suction head, 49 534 lbs.; floating discharge pipe and pontoons, 244 916 lbs.; hull fittings, 31 580 lbs. Total, 1 019 623 lbs. The working draft of the dredge is about 46 ins.

Construction was begun in June, 1896, and the dredge was practically completed in March, 1897.

The results of the official tests are given in Table No. 4.

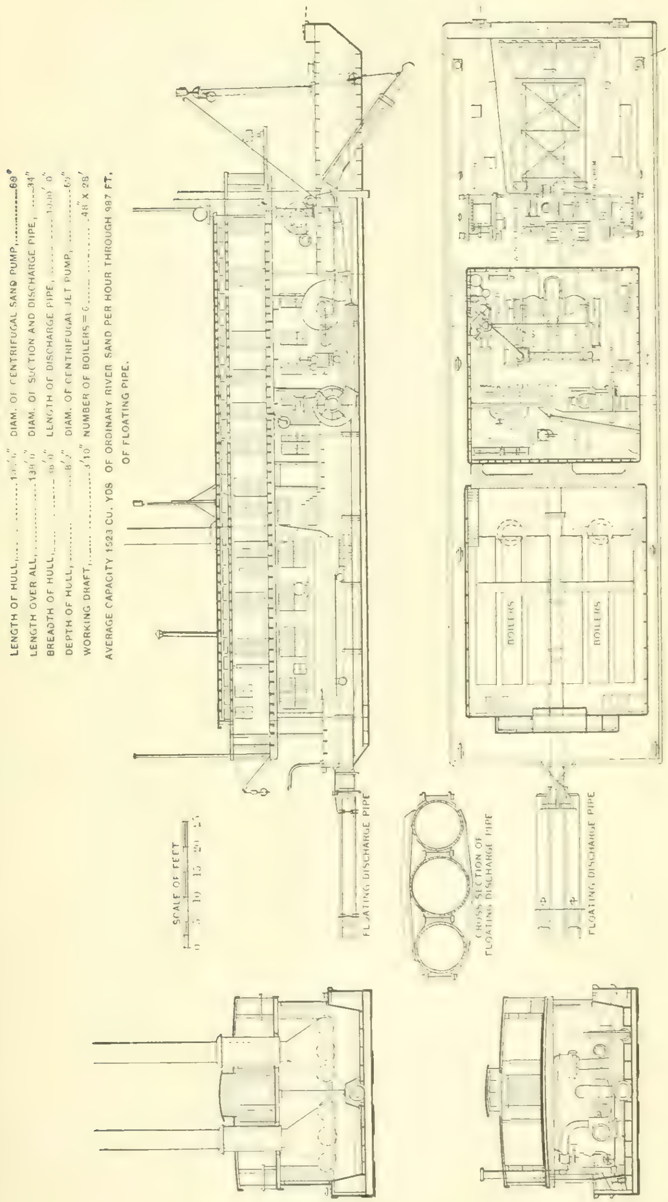

HYDRAULIC DREDGE GAMMA
DIMENSIONS.
LENGTH OF HULL,
LENGTH OVER ALL, 138' 0"
BREADTH OF HULL,
DEPTH OF HULL, 8' 2"
WORKING DRAFT, 3' 10"
DIAM. OF CENTRIFUGAL SAND PUMP, 69"
DIAM. OF SUCTION AND DISCHARGE PIPE, 34"
LENGTH OF DISCHARGE PIPE,
DIAM. OF CENTRIFUGAL JET PUMP,
NUMBER OF BOILERS = 6 48" X 28'
AVERAGE CAPACITY 1523 CU. YDS OF ORDINARY RIVER SAND PER HOUR THROUGH 987 FT. OF FLOATING PIPE.
SCALE OF FEET
0 5 10 15 20 25
FLOATING DISCHARGE PIPE
CROSS SECTION OF FLOATING DISCHARGE PIPE
FLOATING DISCHARGE PIPE
BOILERS
BOILERS

FIG. 10.

TABLE No. 4.—Capacity and Efficiency Tests of United States Dredge *Gamma*.

Number of test.	Duration of test.	Conditions.[1] Depth of water.	Depth of suction.	Forward movement of dredge per minute.	Main Engine and Pump. Steam pressure, pounds per square inch.	Vacuum in condenser, inches of mercury.	Revolutions per minute.	I. H.-P. High-pressure cylinder.	I. H.-P. Low-pressure cylinder.	I. H.-P. Total.	Head. Suction, feet of water.	Head. Discharge, feet of water.	Head. Velocity, feet of water.	Head. Total, feet of water.	Jet Engine and Pump. (Nine jets, 2¼ ins. diameter.) Revolutions per minute.	I. H.-P. High-pressure cylinder.	I. H.-P. Low-pressure cylinder.	I. H.-P. Total.	Head. Suction, feet of water.	Head. Discharge, feet of water.
	Seconds.	Feet.	Feet.	Feet.																
1...	80.4	5.2	14.0	2.0	143	25.5	162	239.8	265.6	505.4	10.3	29.3	2.1	41.7	148	60.4	26.8	87.2	3.7	31.6
2...	90.8	7.0	14.0	3.9	135	25.0	176	212.7	266.6	479.3	8.0	35.0	1.8	44.8	146	56.6	26.0	82.6	4.0	31.6
3...	105.3	4.7	12.0	2.2	152	25.5	174	229.7	256.5	486.2	8.0	38.5	1.1	47.6	143	54.1	23.5	77.6	3.7	31.1
4...	81.2	4.0	11.0	1.8	162	25.0	163	215.6	221.3	436.9	7.5	30.9	2.0	40.4	144	55.3	24.5	79.8	3.7	30.6
5...	59.5	5.2	12.0	4.5	144	25.5	172	242.2	270.0	512.2	9.4	35.0	1.7	46.1	146	55.6	26.4	82.0	3.7	31.3
6...	40.6	4.3	12.0	5.2	139	26.0	165	228.6	249.0	477.6	9.6	38.7	1.5	49.8	144	53.0	24.2	77.2	3.8	33.4
7...	64.7	4.3	12.0	3.5	145	26.5	166	....	....	....	9.2	41.5	1.0	51.7	142	49.4	22.4	71.8	3.8	31.3
8...	67.2	4.8	11.0	5.6	130	26.0	170	225.3	257.3	482.6	8.6	41.5	1.4	51.5	158	63.7	31.0	94.7	4.0	31.6
9...	69.7	5.8	15.0	2.3	140	24.0	166	240.0	240.6	480.6	8.0	33.9	1.8	43.7	160	66.1	32.7	98.8	5.2	34.0
10...	97.6	5.2	15.0	3.2	143	25.0	167	....	....	....	14.3	45.4	0.4	60.1	165	74.1	38.3	112.4	4.2	36.8
11...	64.4	5.2	11.0	6.6	125	26.7	169	225.4	282.1	507.5	10.3	37.3	1.8	49.4	160	68.1	34.7	102.8	4.2	38.2
	74.67	5.06	12.6	3.71	141.6	25.52	168.2	228.81	256.56	485.37	9.38	37.00	1.51	47.89	150.5	59.67	28.23	87.90	4.00	32.86
SPECIAL TEST.																				
12...	63.8	6.3	11.0	2.2	142	24.0	170	239.0	265.2	504.2	8.6	30.4	2.6	41.6	......	......	......	......	......	......

TABLE No. 4—(*Continued*).

Number of test.	Measuring Barge. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.) Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Quantity of material pumped.	Quantity per second.	Velocity of discharge, per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Efficiency. Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, per I. H.-P.	Weight of a cubic foot of dry sand.	Percentage of voids.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu. yds.	Ft.-lbs.	Ft.-lbs.		Cu. yds.	Lbs.	
1..	0.487	2.783	2.296	5 874.8	73.07	11.59	0.332	840.8	14.3	1 394.0	190 438	277 970	68.5	2.76	98.5	37.2
2..	0.421	2.800	2.379	6 089.7	67.07	10.64	0.379	961.5	16.0	1 411.9	187 796	263 615	71.2	2.95	99.0	36.7
3..	0.767	2.967	2.200	5 629.1	53.46	8.48	0.352	892.0	15.8	1 129.5	159 043	267 410	59.5	2.32	105.5	32.8
4..	0.375	2.650	2.275	5 821.0	71.69	11.37	0.187	473.6	8.1	776.2	181 017	240 295	75.3	1.78	99.0	37.3 [2]
5..	0.417	1.967	1.550	3 966.0	66.66	10.57	0.347	878.8	22.2	1 967.4	192 064	281 710	68.2	3.84	103.5	35.7
6..	0.317	1.296	0.979	2 507.5	61.76	9.79	0.214	542.0	21.6	1 780.0	192 228	262 680	73.2	3.73	101.5	34.6
7..	0.367	1.617	1.250	3 198.4	49.43	7.84	0.270	683.8	21.4	1 410.9	159 721 [4]	....	....	....	103.0	34.4 [3]
8..	0.290	1.850	1.560	3 991.6	59.40	9.42	0.436	1 101.0	27.6	2 184.5	191 194	265 430	72.0	4.53	105.0	34.0
9..	0.267	2.117	1.850	4 733.6	67.91	10.77	0.247	625.5	13.2	1 194.6	185 479	264 339	70.2	2.49	99.5	33.2
10..	0.345	1.583	1.238	3 167.7	32.46	5.15	0.263	666.0	21.0	909.6	121 928 [4]	....	....	....	107.0	32.8 [5]
11..	0.379	2.067	1.688	4 319.1	67.07	10.64	0.495	1 253.2	29.0	2 595.2	207 079	279 125	74.2	5.11	106.0	36.3
	0.403	2.1543	1.7514	4 481.68	60.907	9.660	0.3202	810.75	19.11	1 523.09	187 370.9	266 951.7	70.26	3.28	102.5	35.0 [6]
SPECIAL TEST.																
12..	0.533	2.575	2.042	5 224.9	81.89	12.99	0.136	344.0	6.6	718.9	212 914	277 310	76.8	1.43	100.0	36.9 [7]

[1] Suction pipes: Two, 24 ins. diameter; length from mouth to pump = 48 ft. Discharge pipe: Diameter, 34 ins.; length from stern of dredge to valve in barge = 987.1 ft. Length from pump to valve in barge = 1 069 ft. Advance of dredge per revolution of winding engines = $\frac{1}{13}$ ft. [2] Experiment with light load. [3] No indicator cards were taken. [4] Not used in computing average. [5] Pipes sunk at test. No indicator cards were taken. [6] Average. [7] Jets not in operation.

Dredge *Delta*.

The dredge *Delta* was constructed under contract with the New York Dredging Co., which sublet the construction of different parts to various manufacturers and builders of machinery. The contract price for this dredge was $124 940. It differs from the *Gamma* chiefly in having a mechanical agitator instead of jets, in its hoisting and hauling winches, and in the form of pump. Its capacity is also somewhat greater. Its construction was begun in June, 1896, and it was practically completed by the end of June, 1897. After the tests were finished, this dredge was used in removing sand bars below Cairo throughout the low-water season of 1897.

The hull is of steel, 175 ft. long, 38 ft. wide, and $8\frac{1}{4}$ ft. deep. A fender 24 ft. long is carried around the suction, making the boat 199 ft. long over all. The bow and stern of the hull have short rakes, and the midship section is rectangular. The frames of the hull are 22 ins. apart between centers. Angles are used for the deck beams and Z-bars for the floors. There are two longitudinal bulkheads running the full length of the boat, and five cross-bulkheads. They are all made of $\frac{5}{16}$-in. plate, with double angles at top and bottom. All are water-tight and have a syphon in each of the compartments. The thickness of the side plating of the hull is $\frac{3}{8}$ in. and of the other hull plating $\frac{5}{16}$ in. It is all laid fore and aft, is lapped and single-riveted, except on deck, where the seams are planed and butted and fastened with 6-in. butt-straps single-riveted. Cross-seams are similarly butted and riveted.

Special foundations of 12-in. I-beams are provided for sustaining the heavy machinery and boilers. The boat is provided with a cabin 156 ft. long, which protects the machinery and furnishes quarters for the crew. The operating room is at the forward end of the cabin, and is fitted with levers and brakes by means of which the boat is maneuvered.

The dredging pump is different from those on the dredges heretofore described, in the shape of the casing and the runner. The runner has five blades 22 ins. wide, and is 7 ft. in diameter. The edges run close to the casing, but the runner is not concentric with the casing, hence the outer ends of the arms are nearer one side of the casing than the other, the widest space being at the bottom, and the space being nearly cut off by a projection in the casing at the upper

FIG. 1.

FIG. 2.

side of the discharge opening. The shape of the pump runner and the upper half of casing is shown in Fig. 1, Plate XXXIV. The axis of the pump is parallel to the axis of the boat and lies over the center line of the same. The thickness of the sides of the casing is 2 ins., and that of the outer circumference is 3 ins. It is lined on the inner side with steel plates fastened with countersunk bolts. This lining can be renewed when worn out. The shaft of the runner is of forged steel 7½ ins. in diameter and 7½ ft. long. It is fitted to the runner by a taper joint secured by a cap nut and two keys. The shaft has one long bearing through the aft side of the casing. It is provided with water bushing under pressure to keep the sand out of the bearing. The sand pump is driven by a vertical, inverted, two-crank, compound-condensing engine, with 22 and 48-in. cylinders and 24-in. stroke. It is fitted with a piston and slide valve and has an adjustable cut-off for the piston valve. The cylinders are supported by two cast-iron back frames of box section, and four taper-steel front columns 2¾ and 3¾ ins. in diameter. The bed plate is of cast iron, box girder pattern, 8 ft. 9 ins. long by 8 ft. 9½ ins. wide over flanges, and 20½ ins. deep from center of shaft. It has four babbitted journals 13 ins. long and 8½ ins. in diameter. The steam chests are reached by means of a stairway and gallery. The usual accessories in the way of relief valves, drains, lubricators, etc., are provided. This engine was designed to develop 800 H.-P. at 140 revolutions per minute, with a boiler pressure of 160 lbs. and a vacuum of 25 ins.

The air pump and jet condenser has a steam cylinder 12 ins. in diameter, a water cylinder 18 ins., and 24-in. stroke. The condenser is mounted vertically over the water cylinder. The air cylinder is copper-lined 18 by 24 ins., and the plunger is packed with soft packing. The valve seats are brass and the valves are of hard rubber. A brass spray-cylinder throws the water out in jets.

The engine which drives the cutters is horizontal, two-cylinder and non-reversible, all attached to a sliding steel frame, which moves back and forth in guides as the cutter is raised or lowered. This is necessary because the shaft which drives the sprocket chain is not in the axis of motion on which the suction and cutter revolve. The whole engine, with its frame, follows the motion of the shaft, so that the gear and pinion are always engaged. To admit of this motion, the steam pipes are provided with slip joints. The cylinders of this engine are

ins. in diameter and have a 15-in. stroke, with the locomotive type of slide valve. It operates the cutters through spur gears and a sprocket chain.

The fire pump is duplex, with two steam cylinders 8 ins. in diameter and two 5-in. plungers with a stroke of 10 ins. The suction is 5 ins. in diameter, and the delivery 4 ins. This pump furnishes water to all the journals where water pressure is used to keep out sand. The feed pump is also duplex, with two steam cylinders 7 ins. in diameter, two plungers 4½ ins. in diameter, with a stroke of 8 inches.

A pressure filter 7 ft. in diameter, with a capacity of about 65 galls. per minute is provided. A special 3 by 4-in. engine is used to revolve the cleaners.

A 7 by 9-in. vertical engine is provided to run the machines in the repair shop and the electric generator.

There are two winding drums located forward of the sand pump, one on the starboard and the other on the port side of the hull. These are 42 ins. in diameter and 42 ins. long, mounted on a 6-in. shaft with two bearings in the main pillow-blocks carrying a 48-in. gear wheel at one end, and an 84-in. gear wheel at the other. These drums are provided with clutches and brakes, and are driven by two independent double-cylinder, horizontal engines with 10-in. by 12-in. cylinders.

The ladder hoist for raising and lowering the suction, and the spud hoist for raising the spud, each have drums 24 ins. in diameter and 24 ins. long. The former is on the starboard, and the latter on the port side of the boat. The cables from these drums lead to the roof and thence out through sheaves to the ladder and spud. These drums are operated by the same engines that operate the winding drums.

Steam is supplied by four Heine safety boilers rated at 250-H.-P. each. These boilers have two shells 36 ins. in diameter and 19 ft. 4 ins. long; two water legs of flanged and riveted sheets held by hollow stave bolts; 140 lap-welded tubes 3½ ins. in diameter and 16 ft. long. and one submerged movable mud drum in each shell. These boilers are erected in two batteries on the main deck near the aft end of the boat. For each battery there is one stack, 68 ins. in diameter and 64 ft. high above the grate. The feed water is heated by passing through a heater with a shell 3 ft. 1½ ins. in diameter, fitted with 114 corrugated copper tubes 1½ ins. in diameter and 78 ins. long. This heater is situated in the center of the boat between the boilers and the

PLATE XXXV.
PAPERS AM. SOC. C. E.
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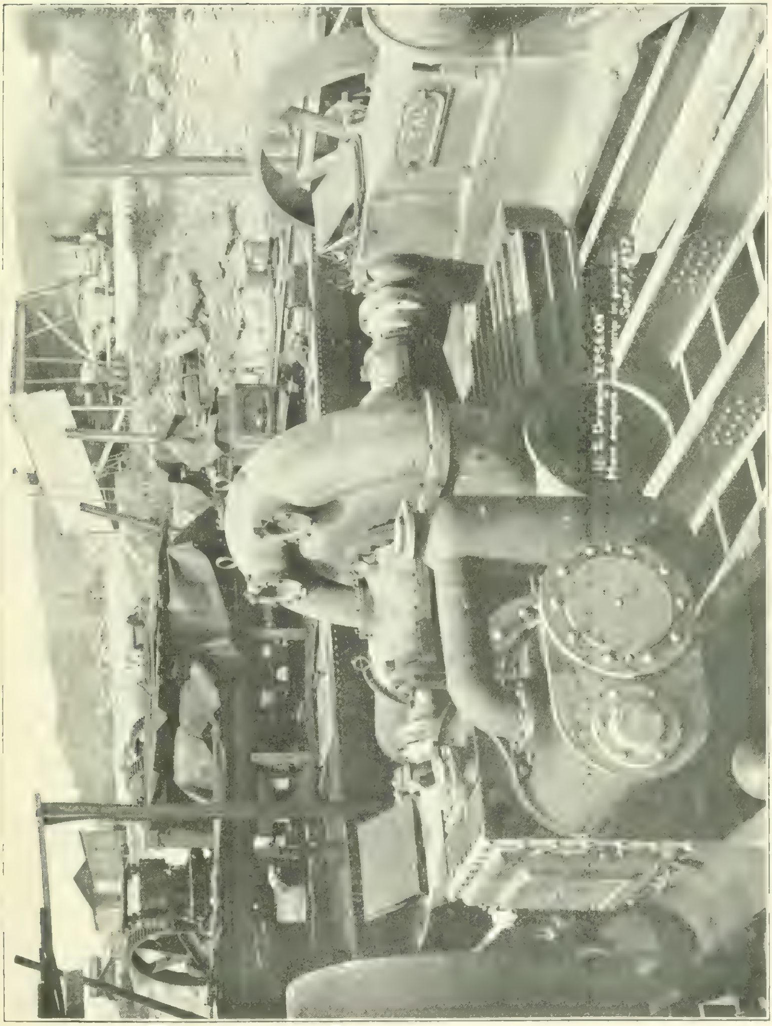
OCKERSON ON DREDGES AND DREDGING.



main engine. The main steam pipe is 6 ins. in diameter, the feed pipe 4 ins., and the drip, blow-off and safety pipes are 2 ins. in diameter.

The intake of the sand pump is from one pipe 34 ins. in diameter, entering in the axis of the pump at the forward side of the casing. This single pipe runs 25 ft. to the forward bulkhead and there branches into two pipes, each $24\frac{1}{2}$ ins. in diameter, which separate and pass through the bow of the boat below the water-line 9 ft. apart. These two pipes turn to the right and left square along the outside of the bow and then turn forward again and each branch separates into two pipes $17\frac{1}{2}$ ins. in diameter, which connect with the suction head. The whole is framed together so that the four pipes, suction head and cutters are raised and lowered together as one piece. Instead of a radial slip joint for the suction pipes, as used on the dredges already described, there is a vertical flanged joint in the horizontal part of each pipe next to the bow, and the revolving pins that sustain the weight of the aft end of the suction are placed in the prolonged axes of these pipes.

The cutter for loosening up the material is placed at the outer end of the suction head. It has twenty-two cast-steel wheel cutters, each having four blades mounted on a steel shaft $6\frac{3}{4}$ ins. square. This shaft is driven by the cutter engine by means of two steel sprocket chains, at a rate of about eight revolutions per minute (see Fig. 2, Plate XXXIV). The discharge outlet is on the starboard side of the casing near the bottom, and leaves the pump at right angles to the intake. After leaving the pump the pipe rises so that the center is slightly above the axis of the pump, and runs aft parallel to the side of the hull until it passes the main engine. Here it bends over to the center line of the hull, and the center of the pipe drops about 11 ins. below the water-line and runs straight aft under the boilers, through the stern, where the floating pipe is coupled on.

The floating discharge pipe is 1 000 ft. long, with the usual rubber couplings at intervals of 50 ft. There are pontoon floats on each side of this pipe, U-shaped in section, with the flat side closed, and they sustain the pipes in yokes which are firmly attached to the floats. There is a baffle plate at the end of the pipe line.

The dredge is provided with sixteen hydraulic piles, six of which are 10 ins. in diameter and 38 ft. long and ten are 6 ins. in diameter and 25 ft. long.

TABLE No. 5.—CAPACITY AND EFFICIENCY TESTS OF UNITED STATES DREDGE DELTA.

Number of test.	Duration of test.	Conditions.[1] Depth of water.	Depth of suction.	Speed of cutters, revolutions per minute.	Forward movement of dredge per minute.	Main Engine and Pump. Steam pressure, pounds per square inch.	Vacuum in condenser, inches of mercury.	Revolutions per minute.	I. H.-P. High-pressure cylinder.	Low-pressure cylinder.	Total.	Head. Suction, feet of water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.
	Seconds.	Feet.	Feet.		Feet.										
1.....	77.0	6.0	11.2	7.7	3.9	159	24.0	150	498.5	536.6	1 035.1	17.0	46.4	4.1	67.5
2.....	62.5	5.8	11.2	9.0	4.4	156	23.3	151	....	....	....	18.7	45.3	4.2	68.2[2]
3.....	71.5	7.0	12.1	7.6	6.0	156	23.0	149	....	....	....	18.1	45.3	3.8	67.2[3]
4.....	79.0	6.7	12.1	9.3	3.8	156	23.0	151	496.7	534.2	1 030.9	19.2	54.5	2.3	76.0[4]
5.....	75.5	8.3	14.2	7.7	2.7	156	23.5	150	491.7	505.2	996.9	18.1	47.6	3.5	69.2
6.....	61.0	5.3	12.5	9.7	4.2	160	23.0	149	532.4	566.2	1 098.6	19.8	45.3	3.4	68.5[5]
7.....	59.25	....	....	....	5.1	161	23.0	152	492.0	490.1	982.1	19.2	47.6	3.0	69.8[6]
8.....	61.0	7.0	12.7	9.4	4.4	160	24.2	149	461.9	456.3	918.2	17.0	47.6	3.7	68.3[7]
9.....	60.0	7.8	13.3	9.2	5.4	161	23.5	150	536.6	553.3	1 089.9	23.2	38.4	3.0	64.6
10.....	60.5	7.0	15.0	7.0	3.7	156	23.3	149	....	....	....	19.2	43.0	3.5	65.7[8]
11.....	58.0	6.3	15.0	8.2	2.5	161	23.0	152	543.1	553.7	1 096.8	18.1	41.8	3.7	63.6
12.....	60.5	10.0	13.5	7.9	4.3	156	23.8	151	515.5	530.3	1 045.8	17.5	43.0	3.3	63.8
13.....	61.0	9.5	14.8	7.4	2.6	156	23.0	154	517.4	554.9	1 072.3	17.5	45.3	3.4	66.2[10]
14.....	72.0	10.0	14.5	7.9	2.8	156	23.0	154	522.3	546.9	1 069.2	18.1	46.4	3.5	68.0[11]
	65.63	7.44	13.24	8.30	3.99	157.9	23.33	150.8	509.8	529.8	1 039.6	18.62	45.51	3.46	67.61[12]
				7.96	3.95	157.6	23.31	151.1	514.6	533.9	1 048.5	18.61	44.55	3.55	66.71[13]
SPECIAL TESTS.															
15.....	53.0	10.5	13.5	9.6	5.2	156	22.8	146	511.0	523.3	1 034.3	21.5	33.8	5.3	60.6[14]
16.....	60.0	8.0	13.3	....	3.8	161	23.2	149	490.4	501.9	992.3	18.1	38.4	3.9	60.4[15]

TABLE No. 5—(*Continued*).

Number of test.	MEASURING BARGE. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.)										EFFICIENCY.					
	Mean depth of water in barge before test.	Mean depth of water and sand in barge at end of test.	Mean depth of material pumped.	Quantity of material pumped.	Quantity per second.	Velocity of discharge per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, per I. H.-P.	Weight per cubic foot of dry sand.	Percentage of voids in wet sand.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu. yds.	Ft.-lbs.	Ft.-lbs.		Cu. yds.	Lbs.	
1....	0.396	3.467	3.071	7 857.8	102.05	16.19	0.398	1 007.9	12.8	1 745.3	430 523	569 305	75.6	1.69	98.5	37.2
2....	0.417	2.958	2.541	6 501.7	104.03	16.50	0.300	759.8	11.7	1 620.9	443 428[9]	…	…	…	76.5	37.2[2]
3....	0.338	3.083	2.745	7 026.2	98.27	15.59	0.471	1 192.8	17.0	2 224.3	412 734[9]	…	…	…	94.5	37.2[3]
4....	0.496	2.850	2.354	6 023.2	76.24	12.09	0.427	1 080.5	17.8	1 823.7	362 140	566 995	63.9	1.77	98.5	34.0[4]
5....	0.383	3.167	2.784	7 123.4	94.35	14.96	0.208	526.8	7.4	928.0	408 064	548 295	74.4	0.93	104.5	35.2
6....	0.571	2.783	2.212	5 659.8	92.78	14.71	0.439	1 111.8	19.6	2 432.4	397 214	604 230	65.7	2.21	108.5	36.3[5]
7....	0.362	2.408	2.046	5 235.1	88.36	14.01	0.518	1 311.8	25.1	2 949.2	385 420	540 155	71.4	3.00	100.5	40.6[6]
8....	0.583	2.900	2.317	5 928.5	97.19	15.41	0.180	455.8	7.7	995.0	414 880	505 010	82.1	1.08	115.5	36.2[7]
9....	0.575	2.633	2.058	5 265.8	87.76	13.92	0.375	950.1	18.0	2 111.3	354 331	599 445	59.1	1.94	96.5	37.1
10....	0.429	2.667	2.238	5 726.4	94.65	15.01	0.576	1 457.7	25.5	3 212.4	388 656[9]	…	…	…	110.5	33.4[8]
11....	0.450	2.650	2.200	5 629.1	97.05	15.39	0.317	802.8	14.3	1 853.2	385 773	603 240	63.9	1.69	100.5	37.5
12....	0.512	2.700	2.188	5 598.4	92.54	14.68	0.263	667.5	11.9	1 471.1	369 003	575 190	64.2	1.41	102.5	38.9
13....	0.342	2.567	2.225	5 693.1	93.33	14.80	0.224	567.3	10.0	1 240.0	386 152	589 765	65.5	1.16	96.5	35.5[10]
14....	0.421	3.100	2.679	6 856.3	95.23	15.10	0.215	544.7	7.9	1 000.9	404 727	588 060	69.0	0.94	112.5	35.4[11]
	0.4482	2.8524	2.4041	6 151.77	93.845	14.883	0.351	888.38	14.76	1 829.12	390 747.9	571 790.0	68.62	1.620	101.14	36.51[12]
	0.4205	2.8545	2.4341	6 228.48	95.238	15.105	0.3514	889.93	14.69	1 850.60	390 499.1	576 681.9	67.88	1.595	99.41	36.81[13]

SPECIAL TESTS.

Number of test.	Mean depth of water in barge before test.	Mean depth of water and sand in barge at end of test.	Mean depth of material pumped.	Quantity of material pumped.	Quantity per second.	Velocity of discharge per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency per centum.	Sand pumped per hour, per I. H.-P.	Weight per cubic foot of dry sand.	Percentage of voids in wet sand.
15....	0.308	2.716	2.408	6 166.5	116.35	18.45	0.242	613.3	9.9	1 542.9	440 675	568 865	77.5	1.49	92.5	33.4[14]
16....	0.467	2.817	2.350	6 012.9	100.22	15.90	0.227	574.0	9.5	1 277.5	378 330	545 765	69.3	1.29	110.5	34.9[15]

[1] Distance from pump to valve in barge = 1 104.5 ft. Distance from end of suction to pump = 84.6 ft. Forward movement of dredge per revolution of winding engines = $\frac{1}{33}$ ft. Ratio of speed of cutters to speed of cutter engine = [illegible]. [2] No indicator cards were taken. [3] No indicator cards were taken. [4] Test barge valve not completely open. [5] Test barge valve did not close and open completely. [6] Cutter machinery broke at test. [7] Test barge valve not completely open. [8] No indicator cards were taken. [9] Not used in computing averages. [10] In sand with a small percentage of gravel. [11] In sand with a small percentage of gravel. [12] Average. [13] Average, excluding tests Nos. 4, 6 and 8. [14] Discharge pipe 532 ft. long, using cutters. [15] Cutters removed from suction.

The various parts of the winding machinery can be handled and moved by means of a 5-ton traveling crane. A traveler is also provided, with which to move the pump and main engine. The repair shop contains a screw-cutting engine-lathe, a drill press, a blacksmith's forge and a full set of tools.

The boat is equipped with one search light, two arc lights and 100 incandescent lamps.

The weights of the various parts of the dredge are about as follows:

Hull	489 500 lbs.
Main pump and engine	105 568 "
Cutter engines and machinery	45 000 "
Winding engines and machinery, with extras	47 000 "
Air, feed and fire pumps, with extras	11 900 "
Electric plant engines, dynamos, etc.	4 275 "
Heater, separator and filter	21 500 "
Derrick, ladder, travelers, tackle sheaves, bolsters, tools, etc	44 000 "
Capstans, spud, piping and fittings, deck fittings, etc.	42 000 "
Discharge and suction pipes pertaining to hull	51 300 "
Boilers, stacks, etc., complete	320 000 "
Cabin, complete	180 000 "
Spare parts for main pump, etc.	13 565 "
Total approximate weight	1 375 608 "

The weight of the floating pipe line is 221 477 lbs.

This dredge was launched on February 20th, 1897, and the tests were completed August 11th, 1897.

The results of the efficiency and capacity tests are given in Table No. 5.

Dredges *Epsilon* and *Zeta*.

The construction of the dredges *Epsilon* and *Zeta* was begun early in January, 1897. They were built under contract with the Springfield Boiler and Manufacturing Company, of Springfield, Ill. The work done by this company at its own shops were confined chiefly to the hulls, floating pipe lines, boilers, and other plate work. The pumps, engines, etc., were sublet to other manufacturers.

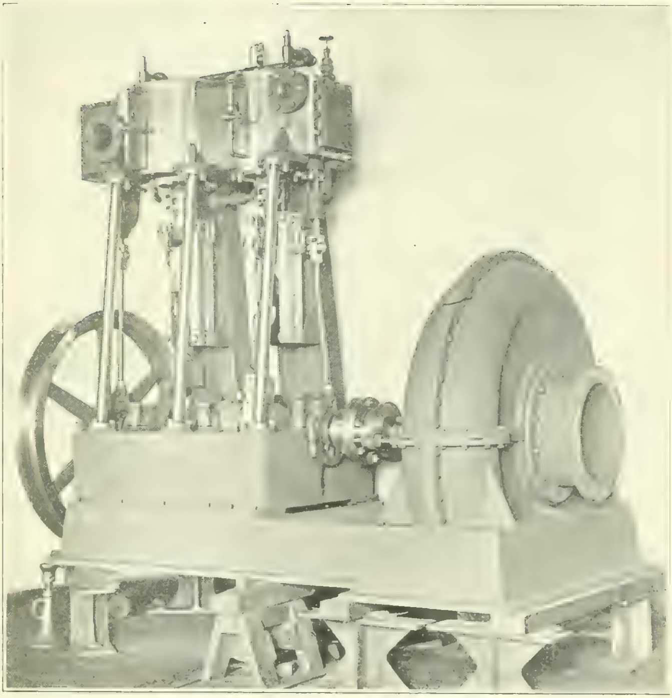


Fig. 1.

Fig. 2.

The stipulated capacity of these dredges is 1 000 cu. yds. of ordinary river sand per hour, dredged from a maximum depth of 15 ft., and delivered through 1 000 ft. of floating pipe.

The dredges are alike throughout, except that the *Epsilon* has water jet agitators and the *Zeta* has mechanical agitators. The contract price of the *Epsilon* is $102 000, and of the *Zeta* $106 000.

The hulls are of steel, 157 ft. long, 40 ft. wide and $7\frac{1}{2}$ ft. deep. There is a well at the bow 35 ft. long, 18 ft. wide at the hull, and $22\frac{1}{2}$ ft. wide at the forward end. There are two open spaces in the hull, one for the engine and pumps, and one for the boiler and coal bunkers, All other parts of the hull are decked over.

The floor beams are 12-in. steel channels, 25 lbs. per foot. The frames are 3 x 4-in. angle irons, 11 lbs. per foot, and are spaced 24 ins. between centers throughout the engine compartment, and 30 ins. for the other parts of the hull. There are ten 15-in. I-beam keelsons, the two outer ones weighing 55 lbs. per foot and the others 45 lbs. per foot. These keelsons are riveted to the floor beams.

The greater part of the hull plating is $\frac{3}{8}$ in. thick. The center strake of the bottom plating, the upper strake of the side plating and the bow end plates are $\frac{3}{8}$ in. thick. The deck plating is $\frac{1}{4}$ in. thick. The butts of all outside plates are planed so as to fit closely. The longitudinal seams are lapped $2\frac{1}{2}$ ins. and single riveted. The plates are butted on the transverse seams and riveted to butt straps $\frac{1}{8}$ in. thicker than the plates.

The hull is divided into eleven water-tight compartments by two longitudinal bulkheads and five cross bulkheads.

The frames in the engine and boiler pits are covered with $\frac{3}{16}$-in. plates. Special beams and angles are provided for supporting the machinery.

The main sand pump is located on the axis of the hull, in the forward part of the engine pit. It is similar in form to that on the *Gamma*. That is to say, it has a divided suction, consisting of two 24-in. pipes, admitting the water on both sides of the casing, and the shaft of the runner extends through the casing with a bearing at each side. The main bearings of the pump runner are protected from sand by means of a ring of water under pressure supplied by the fire pump. The discharge is from the bottom of the casing and is 32 ins. in diameter. The pump runner is 5 ft. 9 ins. in diameter and has seven

blades 11⅝ ins. wide at the outer ends. It is to be run at a speed of about 160 revolutions per minute. Pressure-gauges are provided to show the suction and delivery heads.

The main engines which operate the pump are connected to the ends of the shaft passing through the pump runner by means of flanged couplings. There is a tandem-compound engine at each end of the shaft, each having cylinders 16 and 26 ins. in diameter and an 18-in. stroke. The cranks are set at right angles with one another. These engines are balanced in a horizontal direction, and automatic governors are provided to regulate the speed. They are designed to develop 650 H.-P. at about 180 revolutions per minute, with a boiler pressure of 150 lbs. Plate XXXV show the engine in place. The engine sub-bases connect with and are bolted to the base of the pump casing, thus forming a common base for the whole. Table No. 6 shows the performance of these engines on the *Zeta*.

TABLE No. 6.—Showing Results ot Tests of U. S. Dredge Zeta, at New Madrid, Mo., January, 1898. Size of Engine, 16 Ins. x 26 Ins. x 18 Ins.

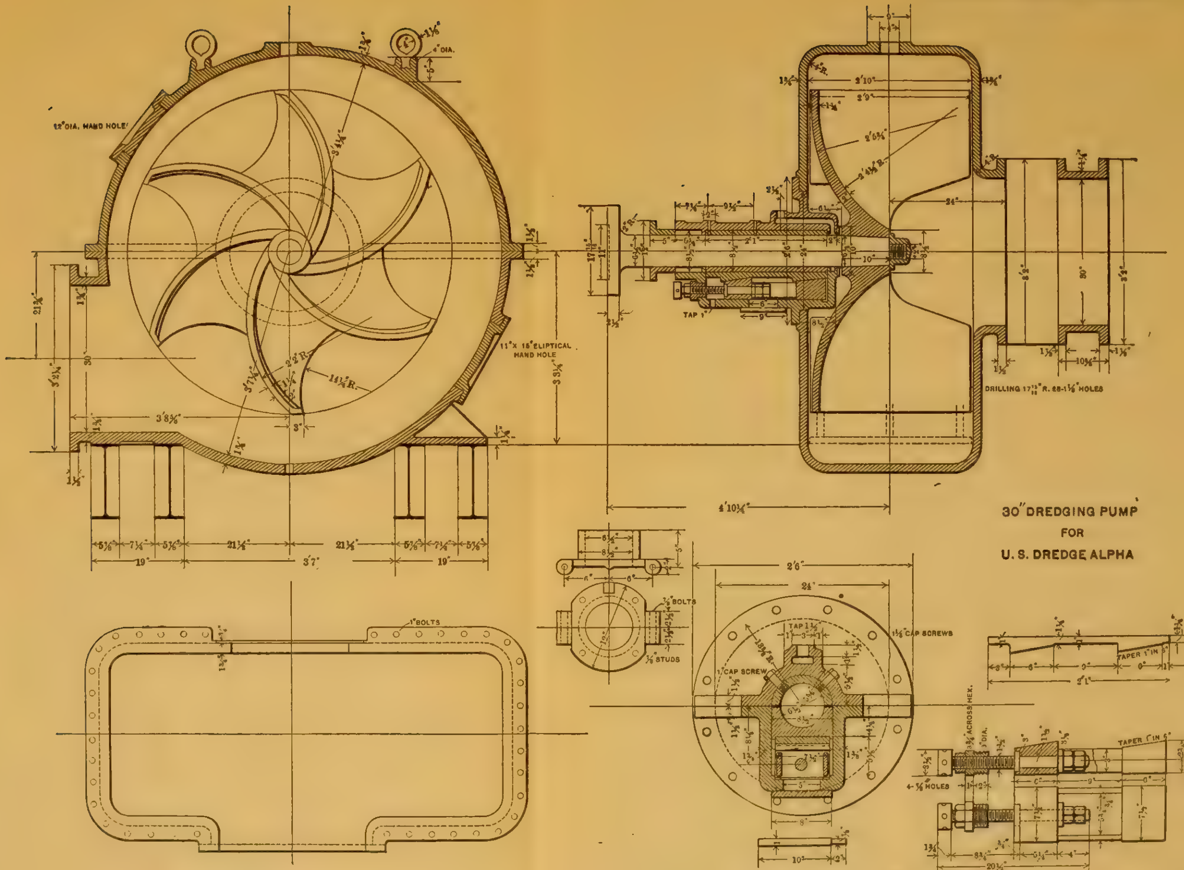
Series.	Revolutions per minute.	Steam.	Suction head, water.	Delivery head, water.	Horse-Power. Starboard engine. High P.	Horse-Power. Starboard engine. Low P.	Horse-Power. Port engine. High P.	Horse-Power. Port engine. Low P.	Total horse-power.	Revolutions of cutter engines.
		Lbs.	Feet.	Feet.						
1........	178	140	11.9	28.5	223	123	174	138	658	...
2........	187	152	11.9	26.5	238	119	207	178	742	...
3........	182	152	11.5	25.5	225	97	195	165	682	...
4........	180	152	13.5	28.5	241	110	204	165	720	147
5........	179	151	12.4	28.5	233	110	197	158	698	120
6........	180	151	12.9	28.5	231	115	191	154	691	135
7........	179	146	14.0	27.5	232	109	195	158	694	105
8........	180	156	12.9	28.5	222	101	184	149	656	156
9........	200	149	15.8	30.7	260	137	272	247	916	...

Notes.—

After taking Card No. 1, the governor was changed to a more sensitive position.

Shaft broke outside of main journal on starboard engine two minutes after cards were taken.

The *Epsilon* has a 15-in. centrifugal jet pump operated by a vertical cross-compound engine with cylinders 12 and 22 ins. in diameter and with 12-in. stroke. The engine is on the same base as the pump, and is directly connected with a flanged coupling. This pump is on the



30" DREDGING PUMP
FOR
U. S. DREDGE ALPHA

starboard side at the rear end of the engine pit. The intake, 18 ins. in diameter, is on the side, and the discharge pipe, 15 ins. in diameter, leaves the casing at the bottom and leads to the pressure chamber at the suction head (Fig. 1, Plate XXXVI). Two boiler feed pumps are located in the engine pit opposite the jet pump, near the port side of the boat. The cylinders are 8 and $4\frac{1}{2}$ ins. in diameter, and the stroke is 10 ins. There is also a fire pump, with cylinders 10 and 5 ins. in diameter, with a 10-in. stroke. These pumps are arranged with suitable cut-off valves so that they can be used together or singly, as may be desired.

The winding engines and the hoisting and hauling drums are located near the bow of the boat. The hauling engines for controlling the movements of the boat are located one on each side of the bow. The drums are 42 ins. long and 42 ins. in diameter, and hold about 1 200 ft. of $\frac{3}{4}$-in. wire rope in two laps. Each of these drums is operated by an 8 by 8-in. double-cylinder engine provided with a link reverse and suitable clutches and brakes giving fast or slow motion, as may be required. The suction hoist is situated a little to the port side of the center line of the hull. The drum is 14 ins. long, and 42 ins. in diameter. It is operated by a double engine with 8 by 8-in. cylinders. The spud hoist is located just behind the anchor spud and on the starboard side of the center line of the hull. It is also operated by an 8 by 8-in. double engine, and is used for raising or lowering the anchor spud. The drum is 21 ins. long and 42 ins. in diameter. The operator stands on an elevated platform behind the winches and controls their movements by means of vertical levers.

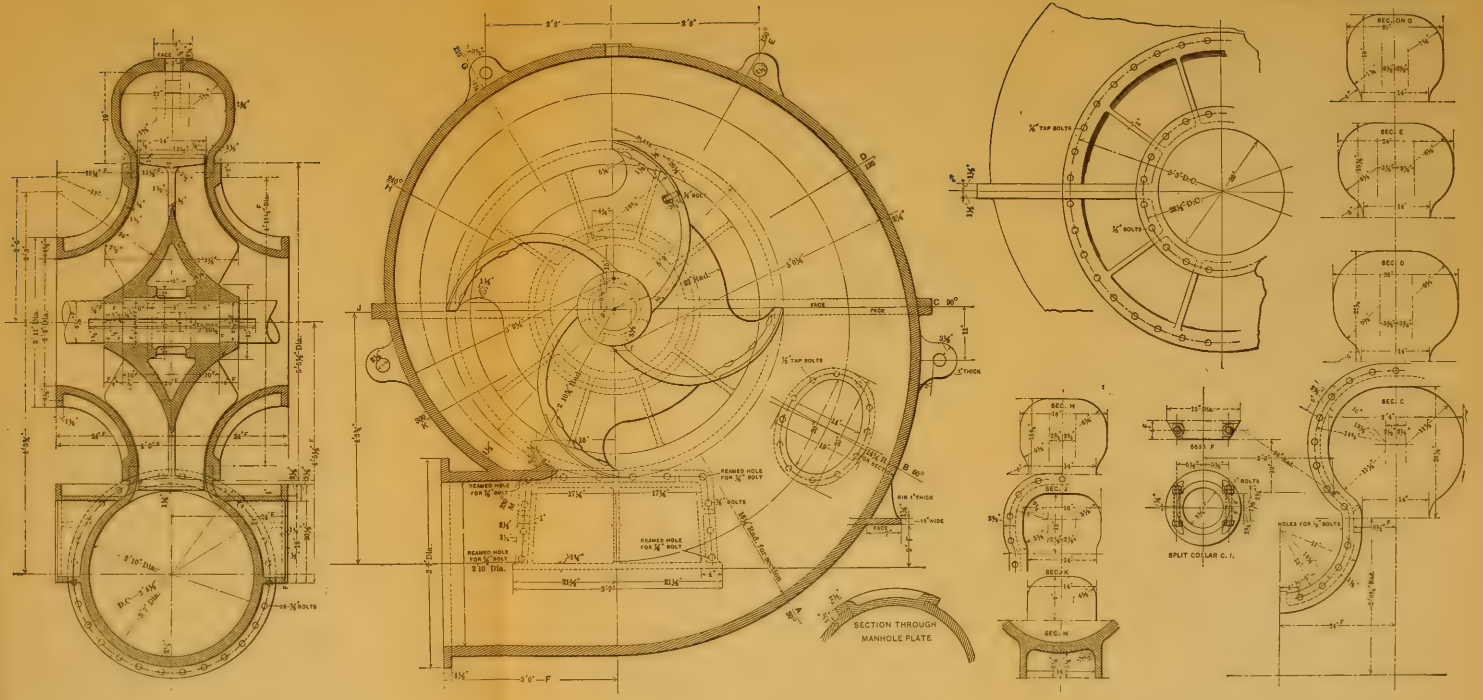
The repair shop is located between the operating room and the engine pit. It is provided with lathe, drill press, screw and bolt cutter, shaper, forge and an engine to run the several machines. A full set of machinist's and blacksmith's tools is also provided. In the same room is located the engine and direct-connected dynamo, furnishing light for all parts of the boat and also a powerful search light at the bow and two arc lights at the stern.

On the *Zeta* the jet pump is omitted and a mechanical agitator for stirring up the material is used. This agitator consists of a vertical scraper or harrow attached to the front end of the suction head which is given an up-and-down motion by means of a bell crank. It is divided into two parts, which are suspended in front of the port and star-

board suctions, respectively. A connecting rod or pitman extends from each agitator back to the rear end of the well, where it connects with a second oscillating crank operated by an engine with 10-in. by 14-in. cylinders. These engines are connected through gearing to a common shaft extending entirely across the boat. In order to lessen the danger of breakage from snags or other solid obstructions, a shearing pin is provided at the angle of the forward crank which is so arranged as to shear off before sufficient strain can come on the agitator to break it. Mechanically considered, its movements are quite satisfactory. In the tests it was demonstrated that the capacity of the pump with this agitator was not much over half the capacity of the pump with jet agitators. It will therefore be taken out, and jets will be put in. This device is the invention of Edward Flad, M. Am. Soc. C. E.

Steam for the various machines described above is derived from six boilers located near the aft end of the boat. These boilers are 48 ins. in diameter and 28 ft. long, with three 11-in. and two 13-in. flues, and are designed for a working pressure of 140 lbs. They are arranged in two batteries so as to be operated separately or together. Each battery has a smokestack 44 ins. in diameter and 70 ft. high. The boilers are set below the main deck and rest on channel bars placed across the keelsons. The feed water is pumped through a heater which receives the exhaust steam from the main engines. A donkey boiler, 48 ins. in diameter and 9 ft. high, is also provided to furnish steam for the electric light and shop engines when the main boilers are not in use.

The two suction pipes run forward from the pump casing, separating gradually until 12 ft. apart at the bow. At this point a tight radial joint is provided, which connects the pipes in the hull with the pipes from the suction head, and admits of the vertical motion necessary for dredging at different depths. The suction can be lowered to a maximum depth of 15 ft. Outside the hull the two suctions are framed together rigidly and are handled as a single piece. The raising and lowering is accomplished by means of sheaves and tackle attached to a derrick frame located over the forward end of the suction well. A straining frame is provided so that when the suction is lowered to a suitable depth it can be locked; it thus relieves the tackle of the weight, and maintains the suction at a constant depth. Each part


SEC. ON G
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SEC. H
SEC. J
SEC. K
SEC. M
SPLIT COLLAR C. I.
SECTION THROUGH
MANHOLE PLATE
REAMED HOLE FOR ⅞" BOLT
⅞" TAP BOLTS
RIB 1" THICK

of the suction head is 10 ft. long, making the suction 20 ft. wide over all.

The *Epsilon* suction head has a pressure chamber on the under side at the front end connecting with the jet pump. Ten 3-in. jet nozzles are screwed into the front side of this pressure chamber and serve to loosen and stir up the sand and induce the flow of material to the suction.

The discharge pipe passes straight aft from the pump between the two central keelsons, and rests on the floor beams for a distance of about 52 ft. From this point it rises gradually for a distance of 22 ft. until the center of the pipe is 4 ft. above the bottom of the boat. Then it runs horizontally straight out through the stern, projecting far enough to couple on the floating pipe line. The pipe is flattened out near the pump casing so as to depress it below the water-line and thus facilitate priming, which is done by using a steam syphon at the top of the casing. The floating discharge pipe is similar to those previously described, except that greater buoyancy is provided, and the shape of the float is somewhat flatter. Fig. 2, Plate XXXVI, shows some of these pipes under construction and gives a good idea of their form. They are built in 50-ft. lengths.

Traveling cranes are provided for the easy handling of the machinery of the engine room and the hoists; and there is one steam capstan at each corner of the dredge. Steam syphons are provided for all the water-tight compartments. Six 10-in. and ten 7-in. mooring piles of the usual form are provided for each dredge.

A velocimeter is provided for showing the velocity of flow in the discharge pipe. This consists of two vertical tubes which pierce the discharge pipe. They are set in the vertical plane passing through the longitudinal axis of the discharge pipe. About 5 ins. below the upper inner surface of the pipe, ¼-in. tubes, 6 ins. long, are inserted horizontally; the one nearest the pump being open toward the pump to receive the pressure of flow and the other turned in the opposite direction. The upper ends of the vertical tubes connect with glass tubes attached to suitable scales. The difference in the height of the columns of water in the two tubes, when water is flowing through the discharge pipe, gives results from which the velocity of flow can be deduced.

Weighing apparatus, designed to show the percentage of solid matter passing through the discharge pipe, is also provided. To ac-

TABLE No. 7.—Capacity and Efficiency Tests of United States Dredge Epsilon. March 14th to 17th, 1898.

Number of test.	*Conditions. Duration of test.	Depth of suction.	Depth of water.	Forward movement of dredge, per minute.	Main Engine and Pump. Steam pressure, pounds per square inch.	Revolutions of main pump, per minute.	I. H.-P. Starboard. High-pressure cylinder.	Starboard. Low-pressure cylinder.	Port. High-pressure cylinder.	Port. Low-pressure cylinder.	Total.	Head. Suction, feet of water.	Discharge, feet of water.	Velocity, feet of water.	Total, feet of water.	Jet Pump. Revolutions per minute.	Head. Discharge, feet of water.
	Seconds.	Feet.	Feet.	Feet.													
1	122.2	15.0	7.8	3.0	153	182	170.6	88.5	228.4	124.1	611.6	13.0	44.7	1.6	59.3	188	22.1
2	102.5	15.0	8.5	3.0	160	181	194.8	101.8	232.4	113.0	642.0	15.8	34.7	3.4	53.9	200	26.7
3	82.6	15.0	8.5	4.4	155	181	176.5	88.0	278.8	169.1	712.4	16.4	34.7	4.4	55.5	191	24.4
4	54.5	14.0	7.3	5.8	155	182	233.0	136.2	260.7	145.8	775.7	18.1	37.7	3.8	59.6	224	31.3
5	61.5	14.0	7.0	9.4	165	182	231.7	137.1	278.7	153.6	801.1	19.8	37.7	4.1	61.6	232	32.5
6	66.4	14.0	8.0	7.1	160	181	239.0	139.8	239.6	119.9	738.3	17.0	37.7	4.3	59.0	228	32.5
7	57.5	15.0	9.0	......	162	182	242.6	137.1	229.8	107.6	717.1	14.7	36.7	5.2	56.6	140	35.6
Average, excluding tests Nos. 1 and 2	64.5	14.4	7.96	†6.67	159.4	181.6	224.56	127.64	257.52	139.20	748.92	17.2	36.9	4.36	58.46	203.0	31.26

* Suction pipes: Two 24 ins. in diameter; length from mouth to pump = 69 ft. Discharge pipe: diameter, 32 ins.; length from stern of dredge to valve in barge = 1 036 ft.; length from pump to valve in barge = 1 121 ft. † Average of Nos. 3 to 6.

Table No. 7—(*Continued*).

Number of test.	Measuring Barge. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.). Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Total quantity of material pumped.	Quantity pumped, per second.	Velocity of discharge, per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Efficiency. Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency, per centum.	Sand pumped per hour, per I. H.-P.	Remarks.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu. yds.	Ft.-lbs.	Ft.-lbs.		Cu. yds.	
1	0.600	3.333	2.733	6 992.9	57.22	10.2	0.274	693.9	9.9	757.1	212 072	336 380	63.0	1.24	Mud lumps in barge, not average material.
2	0.417	3.700	3.283	8 400.2	81.95	14.7	0.217	549.6	6.5	714.9	276 069	353 100	78.2	1.11	Mud lumps in barge, not average material.
3	0.433	3.483	3.050	7 804.0	94.48	16.9	0.419	1 061.1	13.6	1 712.8	327 728	391 820	83.6	2.40	Medium fine sand.
4	0.750	2.617	1.867	4 777.1	87.65	15.7	0.464	1 175.1	24.6	2 874.9	326 496	426 635	76.5	3.71	Medium fine sand.
5	0.300	2.467	2.167	5 544.7	90.16	16.4	0.565	1 430.9	25.8	3 102.2	347 116	440 605	78.8	3.87	Medium fine sand.
6	0.500	2.900	2.400	6 140.9	92.48	16.6	0.555	1 405.5	22.9	2 822.3	341 020	406 065	84.0	3.82	Medium fine sand.
7	0.367	2.667	2.300	5 885.0	102.35	18.3	0.384	972.5	16.5	2 255.1	362 063	394 405	91.8	3.14	Medium fine sand.
Average, excluding tests Nos. 1 and 2.	0.470	2.8268	2.3568	6 030.34	93.424	16.78	0.4774	1 209.02	20.68	2 553.46	340 884.6	411 906.0	82.9	3.388	

TABLE No. 8.—CAPACITY AND EFFICIENCY TESTS OF UNITED STATES DREDGE ZETA. MARCH 21ST TO 24TH, 1898.

Number of test.	Duration of test.	* Conditions. Depth of suction.	* Conditions. Depth of water.	Forward movement of dredge per minute.	Revolutions of cutter engine per minute.	Steam pressure, pounds per square inch.	Revolutions of main pump per minute.	I. H.-P. Starboard. High-pressure cylinder.	I. H.-P. Starboard. Low-pressure cylinder.	I. H.-P. Port. High-pressure cylinder.	I. H.-P. Port. Low-pressure cylinder.	I. H.-P. Total.	Head. Suction, feet of water.	Head. Discharge, feet of water.	Head. Velocity, feet of water.	Head. Total, feet of water.
	Secs.	Feet.	Feet.	Feet.												
1	62.0	14	7.5	4.8	120	145	182	224.5	137.1	197.2	159.6	718.4	13.0	34.7	4.5	52.2
2	60.0	13	7.5	6.0	186	150	182	218.3	120.6	197.5	152.7	689.1	12.4	35.7	3.3	51.4
3	69.6	13	6.0	4.0	155	146	182	225.2	122.3	189.3	141.4	678.2	10.2	35.7	4.2	50.1
4	70.0	14	8.0	3.2	183	146	181	215.4	119.1	186.7	139.8	661.0	10.7	35.7	4.9	51.3
5	61.6	14	8.5	5.0	180	144	180	222.7	129.6	191.5	144.2	688.0	11.3	33.7	4.2	49.2
Average	64.64	13.6	7.5	4.6	164.8	146.2	181.4	221.22	125.74	192.44	147.54	686.94	11.52	35.1	4.22	50.84

* Suction pipes: Two, 24 ins. in diameter; length from mouth to pump = 69 ft. Discharge pipe: Diameter, 32 ins.; length from stern of dredge to valve in barge — 1 036 ft.; length from pump to valve in barge — 1 121 ft.

TABLE No. 8—(*Continued*).

Number of test.	Measuring Barge. (Area of water = 2 558.7 sq. ft.; area of sand = 2 532.5 sq. ft.) Mean depth of water in barge before test.	Mean depth of material in barge at end of test.	Mean depth of material pumped.	Total quantity of material pumped.	Quantity pumped per second.	Velocity of discharge per second.	Mean depth of sand.	Quantity of sand.	Percentage of sand.	Sand pumped, rate per hour.	Efficiency. Work done per second, measured by material pumped.	Work done per second, measured by indicator cards.	Efficiency, per centum.	Sand pumped per hour, per I. H.-P.	Remarks.
	Feet.	Feet.	Feet.	Cu. ft.	Cu. ft.	Feet.	Feet.	Cu. ft.		Cu yds.	Ft.-lbs.	Ft.-lbs.		Cu.yds.	
1..........	0.417	2.717	2.300	5 885.0	94.92	17.0	0.345	873.7	14.8	1 878.9	309 677	395 120	78.4	2.62	Medium fine sand. Sheared four pins before test was obtained.
2..........	0.650	2.567	1.917	4 905.0	81.75	14.6	0.231	585.0	11.9	1 300.0	262 622	379 005	69.4	1.89	Medium fine sand. Test made after shearing four pins, only one agitator running.
3..........	0.317	2.817	2.500	6 396.8	91.91	16.5	0.274	693.9	10.8	1 329.4	287 793	373 010	77.2	1.96	Medium fine sand. Test made after shearing two pins.
4..........	0.433	3.150	2.717	6 952.0	99.31	17.8	0.222	562.2	8.1	1 070.9	318 413	363 550	87.6	1.62	Medium fine sand. Test made after shearing five pins and with only one agitator running.
5..........	0.433	2.650	2.217	5 672.6	92.09	16.5	0.227	574.9	10.1	1 241.4	283 177	378 400	74.8	1.81	Medium fine sand. No pins sheared.
Average...	0.450	2.7802	2.3302	5 962.3	91.996	16.48	0.2598	657.94	11.14	1 364.72	292 336	377 817	77.5	1.98	

complish this, a suitable length of the pipe is connected at each end by rubber thimbles, which leaves the joints flexible. One end of this pipe is then supported by a weighing apparatus properly counter-balanced. An indicator is located where it can be readily seen by the engineer. The pointer is set so that it reads zero on the dial when water only is pumped. When sand is pumped the pointer will show on the dial the weight of the same at any moment, and consequently the percentage of solid matter.

Both dredges have cabins, for the protection of the machinery and to provide quarters for the crew. On the main deck the house is 34 ft. wide, 113 ft. long and 12 ft. high. Above this is the cabin proper, 32 ft. wide, 109½ ft. long and 9 ft. high, built and arranged as in ordinary steamboat practice. There are ample accommodations for a crew of fifty.

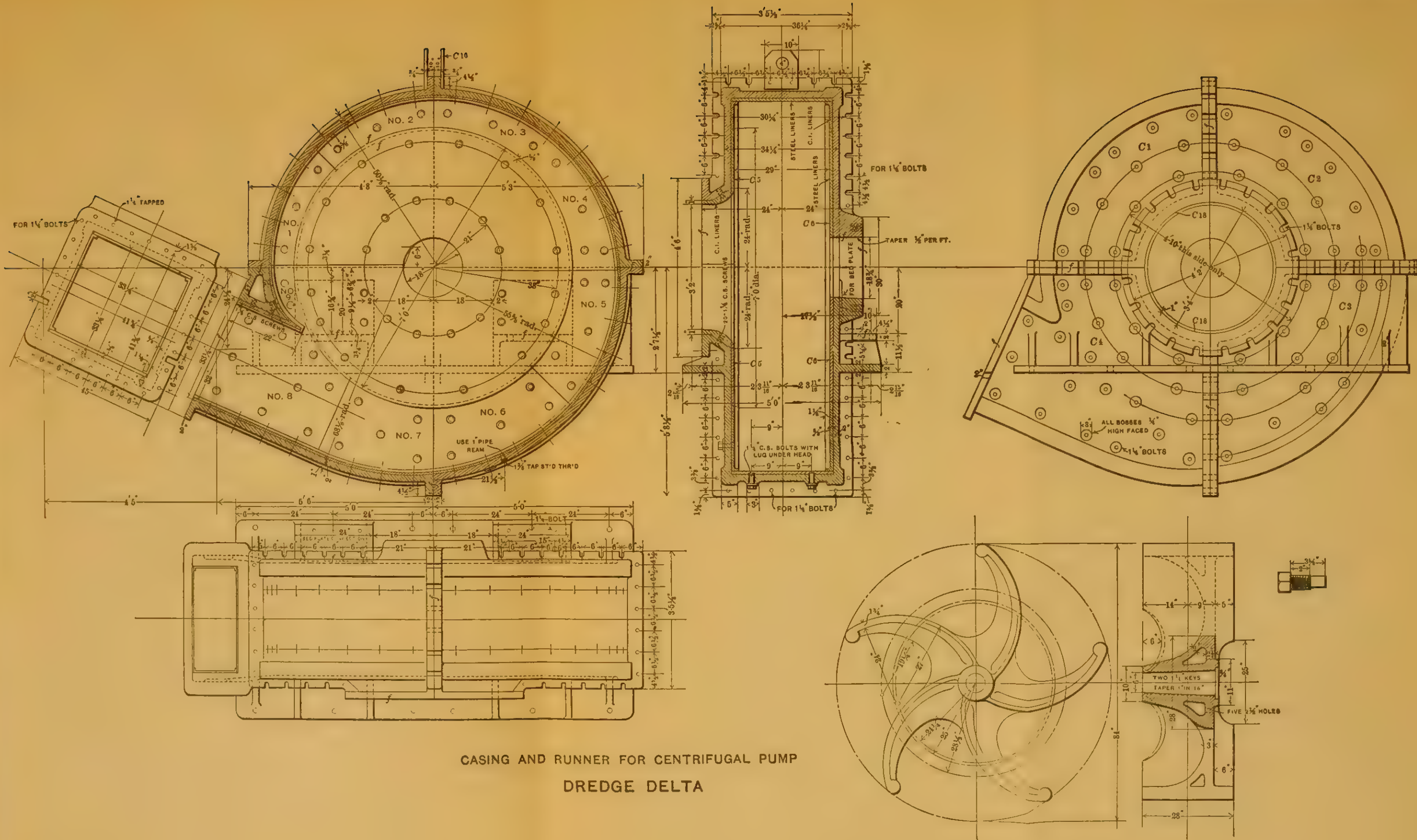
The weights of the various parts of each of these dredges are about as follows: Hull, 442 388 lbs.; main pump and engines, 117 060 lbs.; cutter engine and agitator, *Zeta*, 25 000 lbs.; jet pump and engine *Epsilon*, 19 010 lbs.; winding engines and machinery, 33 436 lbs.; fire and feed pumps, 8 100 lbs.; heater, 4 925 lbs.; electric plant, 4 200 lbs.; derrick, A-frame, travelers, sheaves, pipes and fittings, deck fittings, capstans, etc., 37 500 lbs.; discharge and suction pipes pertaining to hull, *Epsilon*, 54 447 lbs.; same for *Zeta*, 51 509 lbs.; boilers, stacks, etc., complete, 195 000 lbs.; cabin, complete; 120 000 lbs.; total approximate weight of each dredge, 1 057 118 lbs.; weight of floating pipe line, 325 000 lbs.

The efficiency tests of these dredges were begun about the middle of January. Owing to delays occasioned by high water and some minor modifications that were required, the capacity tests were delayed until the latter part of March, 1898.

The results of these tests are given in Tables Nos. 7 and 8.

The specifications and plans of all the dredges of the Mississippi River Commission have been prepared under the direction and subject to the approval of the Committee on Dredges, consisting of Major Thomas H. Handbury, Corps of Engineers, U. S. A.; Henry Flad, M. Am. Soc. C. E., and B. M. Harrod, Past-President, Am. Soc. C. E.

The dredges *Gamma*, *Delta*, *Epsilon* and *Zeta* were constructed under the direction of Captain H. E. Waterman, Corps of Engineers, U. S. A.



CASING AND RUNNER FOR CENTRIFUGAL PUMP

DREDGE DELTA

TABLE No. 9.—PRINCIPAL FEATURES OF THE U. S. DREDGE BOATS OPERATING ON THE MISSISSIPPI RIVER BELOW CAIRO, ILL.

Name of Dredge.	*Alpha.*	*Beta.*	*Gamma.*	*Delta.*	*Epsilon.*	*Zeta.*
When built	1893.	1895.	1897.	1897.	1897-98.	1897-98.
Cost	$87 000	$217 000	$86 000	$125 000	$102 000	$106 000
Capacity in cubic yards of sand per hour	600	2 000	800	1 000	1 000	1 000
Hull—Kind	Wood.	Steel.	Steel.	Steel.	Steel.	Steel.
Size	140' x 36' x 8'	172' x 40' x 7' 2"	138' x 38' x 8'	175' x 38' x 8' 4"	157' x 40' x 7' 9"	157' x 40' x 7' 9"
Working draft	4' 10"	6' 6"	4' 3"	4' 3"	4'	4'
Main engines—Number	1	2	1	1	2 coupled	2 coupled
Kind	Cd. non-cond.	4 cyl. trpl. exp. 2 crank cond.	Horiz. cross comp. cond.	Vertical comp. cond.	Horiz. tandem coup. non-cond.	Horiz. tandem coup. non-cond.
Size	15" 27" x 20"	20½"-33"-38"-38" x 24"	18"-22½" x 22"	24" 48" x 24"	16"-26" x 18"	16" 26" x 18"
Horse-power	300	2 000	500	800	650	650
Revolutions per minute	140	135	150	150	180	180
Kind of condenser	None.	Jet.	Jet.	Jet.	None.	None.
Main dredging pumps—Centrifugal	1	2	1	1	1	1
Diameter of runner	6'	7'	5' 9"	7' x 28" wide.	5' 9"	5' 9"
Kind of suction	Side.	Divided.	Divided.	Side.	Divided.	Divided.
Diameter of suction	30"	Six 19½-in.	Two 24-in.	Four 17½-in.	Two 24-in.	Two 24-in.
Discharge pipes—Number	1	2	1	1	1	1
Discharge pipes—Size	30"	33"	34"	34"	32"	32"
Suction head in feet of water	7	18	9	18	12	12
Delivery head in feet of water	20	29	37	45	28	28
Velocity—Feet per second	10	14	10	15	13	13
Sand agitators	6 jets 2½" diam.	6 vert. cutters.	9 jets 2½" diam.	Horiz. cutters.	10 jets 3" diam.	Oscillating rake.
Engines, kind	Cd. non-cond.	Cross cd. non-cond.	Comp. cond.	2 simple.	Vert. comp. non-cond.	2 simple.
Engines, size	....	14½" 29" x 18"	12"-22" x 14"	12¼" x 15"	12"-22" x 12"	Each 12" x 16"
Engines, revolutions per minute	190	120	160	....	200	180
Cutters, revolutions per minute	....	12	....	5	....	60 double strokes
Engines, horse-power	125	....	200	....	150	80
Jet pumps	Centrifugal.	....	Centrifugal	....	Centrifugal.	....
Suction and discharge diameter	10"	....	18"	....	18" and 15"	....
Suction head	6	....	7	....	5	....
Delivery head	20'	....	28'	....	30'	....
Boilers—Number	4	4	6	4	6	6
Kind	Miss. River.	Heine 375 H.-P.	Miss. River.	Heine 250 H.-P.	Miss. River.	Miss. River.
Size	42" x 28'	....	48" x 28' Five 11" flues.	....	48' x 28' Three 11" & Two 13" flues.	48" x 28" Three 11" & Two 13" flues.
Pressure used	135 lbs.	165 lbs.	140 lbs.	155 lbs.	150 lbs.	150 lbs.
Coal used per 24 hours	500 bu.	2 088 bu.	400 bu.	1 200 bu.	....	....
Total cost of running dredge per day	$98.70	$221.63	$100.51	$111.76	....	....

Cost of steam tender per day, $37. Cost of pile sinker per day, $13.

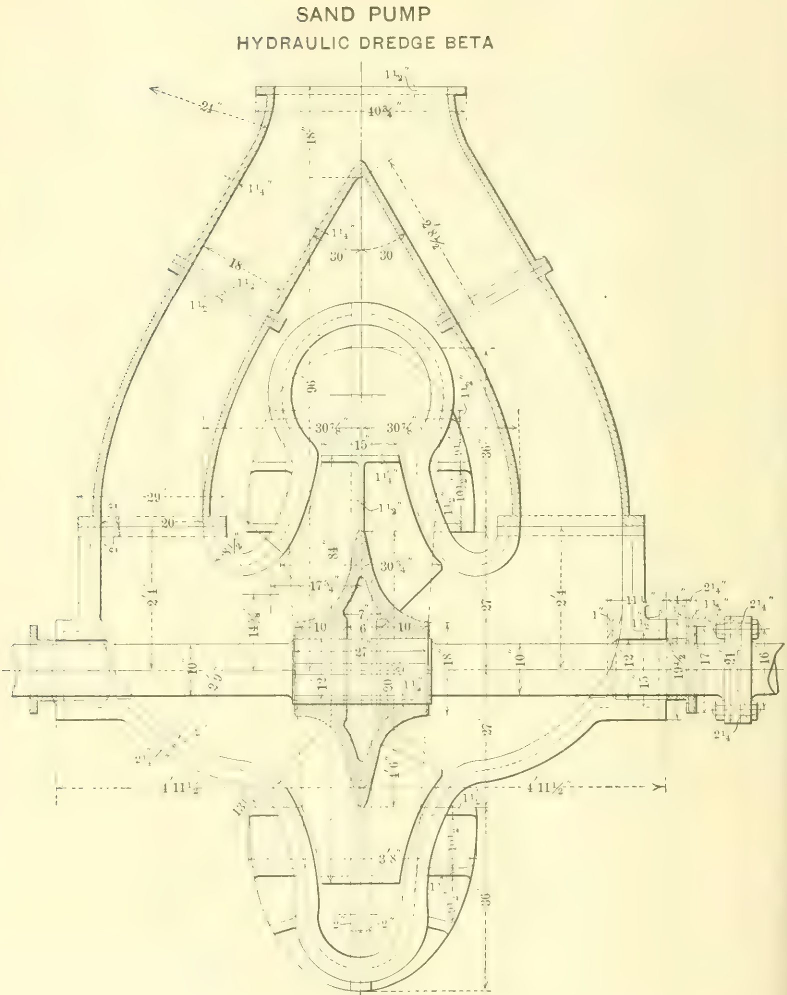


FIG. 11.

SAND PUMP

HYDRAULIC DREDGE BETA

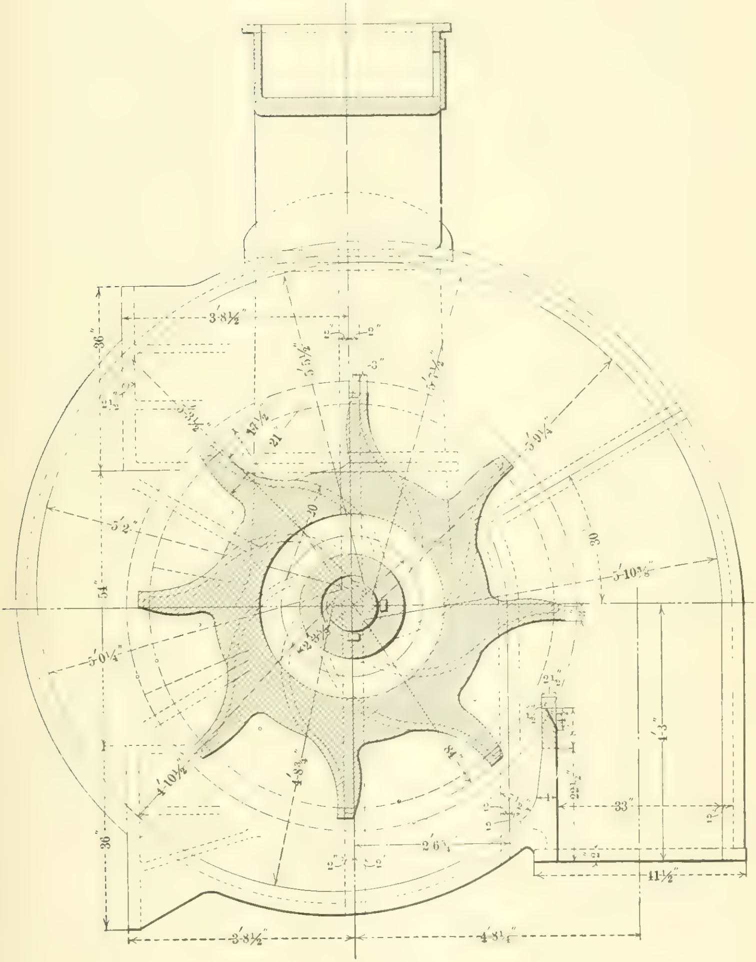


FIG. 12.

Another dredge has been designed for work on the river below Cairo, and the contract for its construction will be let during the present year. This will be a single-pump dredge, similar in form to the *Epsilon*, but will be provided with side wheels and propelling machinery.

Perhaps the most important feature of a hydraulic dredge is its pump. As there is a wide variance in practice as to the form and size of pump runners, it will probably be interesting to show the details of the several pumps now in use on the Mississippi River. These are shown in the following plates: *Alpha*, Plate XXXVII; *Beta*, Figs. 11 and 12; *Gamma*, Plate XXXVIII; *Delta*, Plate XXXIX; *Epsilon* and *Zeta*, Plate XL.

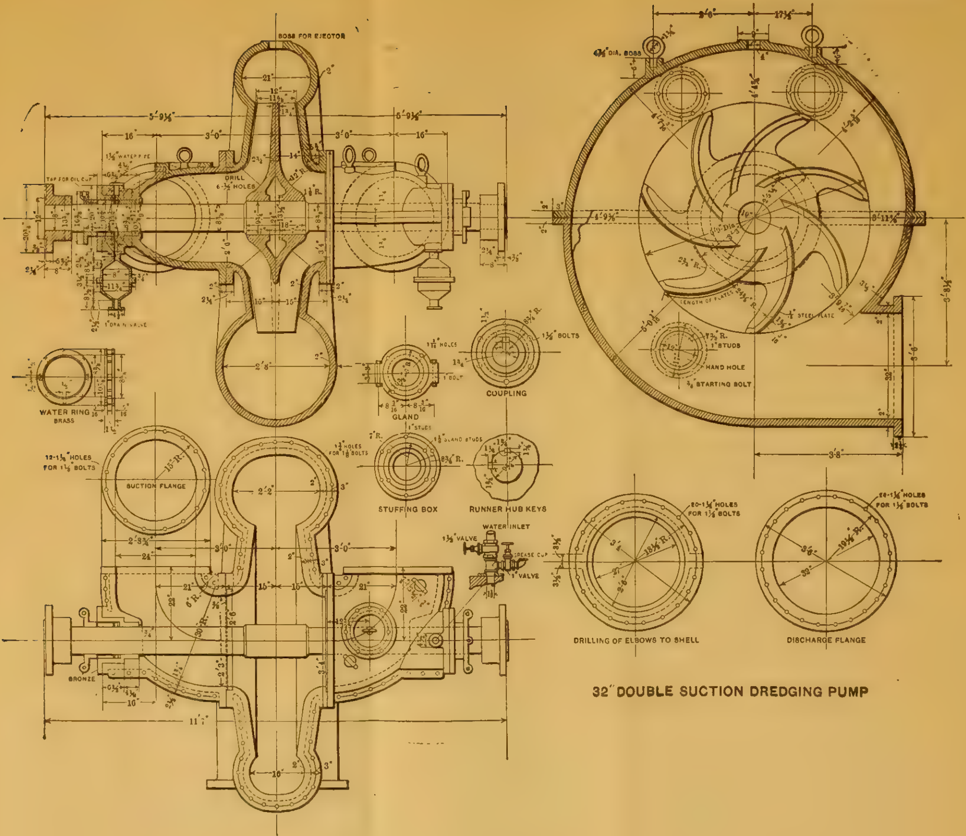
The general features of the dredges now in use on the Mississippi River below Cairo are summarized in Table No. 9.

Two dredges for use on the Mississippi River between the Missouri and Ohio Rivers are now under construction and are practically completed. They were designed and constructed under the direction of Major Thomas H. Handbury. They will be ready for use during the next low-water season. It is expected that a navigable channel 6 ft. deep can be maintained with these dredges, aided by the temporary jetties and the jet dredge.

The hulls of these dredges are of steel, 160 ft. long and 40 ft. wide. with $6\frac{1}{2}$ ft. depth of hold. Their working draft will be about $3\frac{1}{2}$ ft. The framing of the hulls consists of 12-in. channels weighing 25 lbs. per foot for floor beams; side frames 3 x 4-in. angles, 11 lbs. per foot, riveted to web of floor beams. The above frames are spaced 12 ins. in the engine and boiler pit, and 30 ins. through the remainder of the hull. There are 10 keelsons, running the entire length of the boat, made of 15-in. steel I-beams. The outboard beams weigh 55 lbs. per foot and the intermediates 45 lbs. per foot. Each keelson is riveted to each floor beam. The center strake of the bottom plating is $\frac{3}{8}$ in. thick, the other strakes being $\frac{5}{16}$ in. thick. The deck plating is $\frac{1}{4}$ in. thick. The upper strake of the side plating is $\frac{3}{8}$ in. thick, and the other side plates are $\frac{5}{16}$ in. thick. The hull is divided into 11 water-tight compartments.

Each dredge is provided with two centrifugal pumps with a single suction in the axis of the pump discharging from the lower side of the casing. The suction is 20 ins. in diameter and the discharge at the

PLATE XL.
PAPERS AM. SOC. C. E.
JUNE, 1898.
OCKERSON ON DREDGES AND DREDGING.



32″ DOUBLE SUCTION DREDGING PUMP

pump is the same diameter, but is expanded to 24 ins. at the discharge pipe. The casing of the pump is 48 ins. in diameter and 28 ins. across the inside. It is provided with liners which can be renewed when worn. The pump runner is 48 ins. in diameter and is provided with three reversed curved arms. The ends of these arms are fitted with detachable extension plates made of mild steel. An adjustable thrust bearing is provided. A water-jet is connected with the main bearing to keep out the sand and grit when the pump is running.

Each pump is operated by a direct-connected, horizontal, compound, non-condensing engine. These engines are designed to run 200 revolutions per minute and develop 300 H.-P. at a boiler pressure of 140 lbs. This speed is to be regarded as the average work of the engine, and is expected to maintain through the suction and discharge pipes the velocity necessary to carry at least 20% of sand and 80% of water under a maximum head of 30 ft.

A jet pump is provided with each main pump which serves to loosen up the material at the end of the suction and thus facilitate its passage into the suction pipe. The jet pump has two high-pressure cylinders 8 ins. in diameter, two low-pressure cylinders 12 ins. in diameter, and two water plungers 12 ins. in diameter; all having 10-in. stroke. The capacity of the pump is 1 200 galls. per minute against a water pressure of 65 lbs. when running compound under a steam pressure of 150 lbs. These pumps discharge through 8-in. pipes terminating in three bronze nozzles 1½ ins. in diameter, their extremities radiating to a distance of about 30 ins. apart and just beneath the suction of the main pump.

Steam is supplied by six boilers of the Mississippi River steamboat type. These boilers are 48 ins. in diameter and 28 ft. long, and each shell contains five 11-in. flues. They are made of marine steel of 60 000 lbs. tensile strength, and in all respects conform to the rules of the Board of Supervising Inspectors. They are designed for a working pressure of 140 lbs. per square inch. The boilers are arranged in two batteries, with three boilers in each, which are connected by one 30-in. steam drum and two 15-in. mud drums. The steam connections are so arranged that one or both batteries can be used at will. There are two smokestacks 42 ins. in diameter and 70 ft. high. The boilers are supplied by two feed pumps with outside-packed plungers 4½ ins. in diameter and 10-in. stroke.

The boilers and machinery rest on I-beam keelsons, the hull being left open for the space occupied by these parts. The remainder of the hull is decked over with steel plates. A vertical anchor spud is placed near the bow of the hull.

The main suction pipes are suspended from a derrick frame and are raised or lowered as required by winches, so that each suction can be operated independently. The suction head opening, or end of the suction pipe, is 20 ins. high by 120 ins. long. The inner end of the suction works on a hinge and slip joint.

There is one 24-in. tank-steel discharge pipe for each pump. Each of these pipes is attached to the fixed sections in the hull by a swivel joint which admits of swinging the pipe out sidewise. The pipes are in lengths of 24 ft., and each length is floated on a steel pontoon with pointed ends, the pipe being attached to the float at the middle joint in such a way that the pontoons lie parallel with the current when the pipes are deflected to one side. Each discharge pipe is 500 ft. long, the several joints being connected together with rubber gaskets. The first joint is connected to the fixed joint at the stern of the dredge by a heavy rubber coupling 3 ft. 8 ins. long.

In operating the dredge, two iron piles are set on line with the cut, and the dredge is attached to them by two $\frac{3}{4}$-in. wire cables 1 200 ft. long. The dredge end of each of these cables is attached to a winch having a 42-in. drum. These winches have double engines and are geared so as to give speeds from $\frac{1}{4}$ in. to 7 ins. per second. The dredge is pulled ahead as rapidly as the dredged material can be taken through the pipes, which, of course, varies with the depth of material handled.

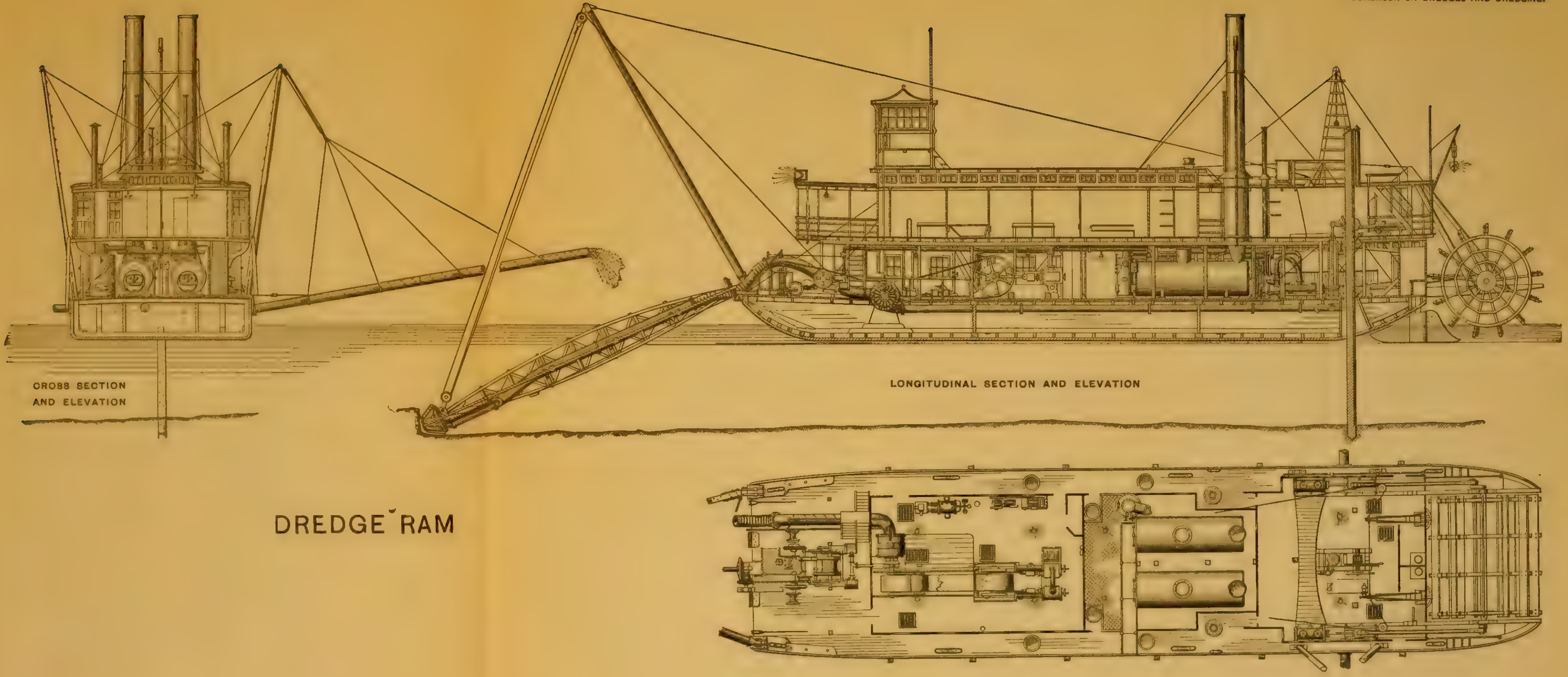
The electric light plant consists of one 4 000 candle-power search light, two 2 000-candle-power arc lights, and seventy-five 16-candle-power incandescent lights. The dynamo is a 110-volt machine of the Western Electric pattern, operated by a horizontal, single expansion 7 x 10-in. engine.

A cabin provides ample quarters for officers and crew, and a machine shop gives facilities for repairs.

Dredge Ram.

In the fall of 1893 the dredge *Ram* was completed by the Bucyrus Steam Shovel and Dredge Company, for use in keeping open the channel from the Mississippi River into the Red and Atchafalaya Rivers.

PLATE XLI.
PAPERS AM. SOC. C. E.
JUNE, 1898.
OCKERSON ON DREDGES AND DREDGING.



It was built under the direction of Captain John Millis, Corps of Engineers, U. S. A. This dredge cost $69 500. The capacity is about 300 cu. yds. of mud per hour, delivered through 300 ft. of pipe. It is provided with propelling machinery such as commonly used on stern-wheel steamboats. The hull is 95 ft. long, 27 ft. beam and 7½ ft. deep. It is built of long-leaf yellow pine. It has two longitudinal and two transverse bulkheads, dividing the hull into nine water-tight compartments. The dredging pump is an Edwards centrifugal, with 15-in. suction and discharge. The intake is from the starboard side and the discharge from the bottom of the casing. The pump is run through a belt connection by a horizontal, compound-condensing engine. The cylinders are 14½ and 26 ins. in diameter, with a stroke of 20 ins.

This dredge can work in any depth up to 30 ft. It is so arranged that material can be discharged on either side at will. A vertical anchor spud is provided in the stern of the boat, on which it swings by means of kedge anchors laid out from either side of the bow and maneuvered by winding drums. In this way a wide cut can be made by swinging the cutters from side to side.

The cutter and suction is supported by an A-frame, the inner legs of which are pivoted to the sides of the bow. The frame is raised or lowered with wire rope tackle attached at the upper end to suitable shears and operated by winding drums. The cutter is a conical steel casting with eight steel blades forming a cutter 24 ins. in diameter at the end and 54 ins. in diameter at the base. This cutter is at the outer extremity of a 5-in. steel shaft, which is revolved by gear wheels at the bow of the boat, and operated by a sprocket wheel and chain. The suction proper starts at the base of the cutter.

The propelling engines have 15-in. cylinders and a 52-in. stroke. There is a combined, duplex air and feed pump and condenser, with 7½-in. steam cylinder, 7-in. air cylinder and 10-in. stroke. Fire and bilge pumps are also provided.

The cutter shaft and winding drums are operated by a pair of auxiliary engines, the cylinders of which are 9 ins. in diameter and 12-in. stroke. The drums are worked by separate clutches.

Steam is supplied by two main boilers 16 ft. long and 60 ins. in diameter. Each shell has 74 3-in. tubes and a corrugated furnace flue 40 ins. in diameter and 72 ins. long.

The boat is lighted throughout by electricity. There is a double-deck cabin which provides ample accommodations for the crew and protects the machinery.

This boat has been quite successful from the start. During the low-water season of 1894 she dredged 265 000 cu. yds. of material at a cost of a trifle over $4\frac{1}{2}$ cents per yard. The material was largely mud and clay. The material is generally delivered through about 50 ft. of pipe suspended from a derrick and attached to the boat, as shown in Plate XLI, from which a good general idea of this boat can be obtained.

It has also been used with considerable success in building levees. It is estimated that under favorable conditions a levee 10 ft. high can be built at a cost of about 6 cents per cubic yard.

Menge Dredges.

In 1888 two dredges of the elevator type were procured, one for use in Vicksburg harbor and one for the mouth of the Red River. These were called the *Menge* and the *Pah-Ute*.

In actual work the *Menge* has dredged 4 000 cu. yds. in 10 working hours. This dredge cost about $18 000. The *Pah-Ute* has about half the above capacity and cost $10 000. These dredges, as generally equipped, deliver the dredged material into dump scows which are towed to the desired point and dumped. In some favorable situations the material is delivered through sluice boxes supported on barges. These dredges, if properly constructed, are very successful when operated in soft material.

At the mouth of the Red River the efficiency of propeller wheels and stern-wheel boats have been thoroughly tried in removing the sediment and keeping the channel open. Under favorable conditions some good results have been obtained in this way; but on the whole, it is generally conceded to be a very poor makeshift.

Dredge Bayley.

The dredge *G. W. R. Bayley* was built by Carroll & Company, of Pittsburg, for use in South Pass. It arrived at Port Eads in the fall of 1877. This boat is constructed of iron throughout. The hull is about 200 ft. long over all, 32 ft. wide, and 10 ft. depth of hold. With fuel on board, it draws about 5 ft. of water. It is self-propelling

Fig. 1.

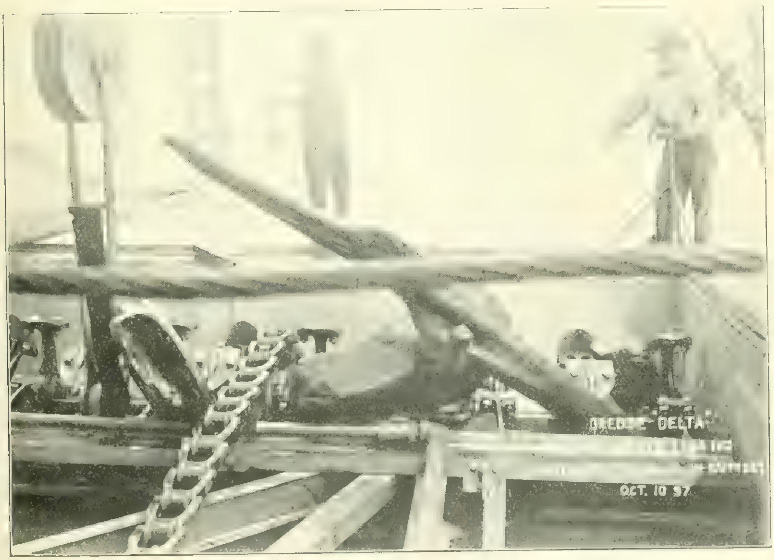


Fig. 2.

by means of side wheels 25 ft. in diameter, with buckets 11 ft. long. These wheels are operated by two engines with 21-in. cylinders and 7-ft. stroke. The boat is provided with a rudder at each end to obviate the necessity of turning when dumping or returning to the cut.

This dredge has a centrifugal pump of the Andrews' Cataract pattern, with a runner 6 ft. in diameter and 3 ft. wide, mounted on an 8-in. steel shaft. It is operated by two independent condensing engines, placed forward of the pump, having 24-in. cylinders and 20-in. stroke. The wrought-iron suction pipe is 27 ins. in diameter. This suction operates in a recess 4 ft. wide, and 25 ft. in length, located at the stern of the boat. It connects with the pipe in the boat, just above the bottom plating and near the pump, by means of a joint which admits of a deflection of 30° from the axis in any direction. The lower end of the suction is curved and flattened out to a width of 4 ft., and is armed with a steel scraper. The suction is supported on a flat plate resting on the sand to prevent the scraper from cutting too deep. This scraper takes in a slice of sand about 8 ins. deep and 4 ft. wide. A stream of water of about half the above depth and the same width enters the suction at the same time.

The movable part of the suction, when in operation, in 26 ft. of water, stands at an angle of about 35 degrees. When not in use it is lifted above water, under the extension of the main deck, the end projecting about 15 ft. aft of the stern post. The discharge pipe is 30 ins. in diameter and is arranged so that the spoil can be deposited in the spoil bins on board, or delivered on either side of the boat through pipes swung from derricks. There are four boilers, 42 ins. in diameter and 26 ft. long, with four flues in each.

The four spoil bins stand forward of the engines. They are 60 ft. long, 19 ft. wide and about 20 ft. deep, and have a capacity of about 512 cu. yds. of dredged material. These tanks are filled in about 7 minutes, when the suction is raised and the boat proceeds to the dumping ground. Two-thirds of the available time is occupied in unloading. Overflow gates are provided so as to allow the water to escape while the sand settles to the bottom, thus insuring a greater load of solid matter. In a working day of 13 hours the average amount of solid matter removed is 1 309 cu. yds. The tanks are divided into compartments, each of which terminates in a hopper having a 4-ft. opening through the bottom of the hull. These openings are

closed by valves operated by hydraulic jacks with a capacity of about 12½ tons. The sand is washed out of the tanks by means of two jets in each hopper.

The capacity of the dredge when delivering the material as described is about 350 cu. yds. per hour. If the material is simply delivered overboard, the capacity is about 1 000 cu. yds. per hour. When dredging, the boat moves down stream, with the suction lowered, at the rate of about 2½ miles per hour.

This dredge cost about $150 000. It was designed by Jas. B. Eads, M. Am. Soc. C. E., for the special purpose of working in deep rough water with strong currents.* This boat is now over twenty years old, but is still serviceable.

The foregoing descriptions cover a fleet of eight hydraulic dredges of large capacity and five other dredges, all of which are now available for use in improving the low-water navigation of the Mississippi River. The dredge *Alpha* has been operated for three seasons, the *Beta* for two seasons, and the *Gamma* and *Delta* for one season. They have all demonstrated the practicability of moving economically immense quantities of material in a short space of time. They have also shown that good navigable channels can be opened through the reefs that obstruct navigation.

As far as the machines themselves are concerned, they seem to meet the requirements fully. The value of these dredges as aids to navigation depends far more on the proper location and direction of the cuts to be made than on the efficiency of the machinery. Experience may show that in some cases the dredges can be made more efficient by filling side chutes and thus throwing the water into one channel, rather than attempting to enlarge one of the channels by dredging out the material. It may also be found desirable in some cases to discharge the material at the sides of the proposed channel rather than to deliver it at a distance through long discharge pipes.

As has already been stated, the volume of material rolled along the bottom is so great as to far outweigh the greatest possible capacity of any dredge. To those who are not familiar with this phenomenon it will seem incredible that a dredge with a capacity of over 1 000 cu. yds. per hour may be operated for days and leave scarcely a trace of

* See "The Mississippi Jetties," by E. L. Corthell, M. Am. Soc. C. E.

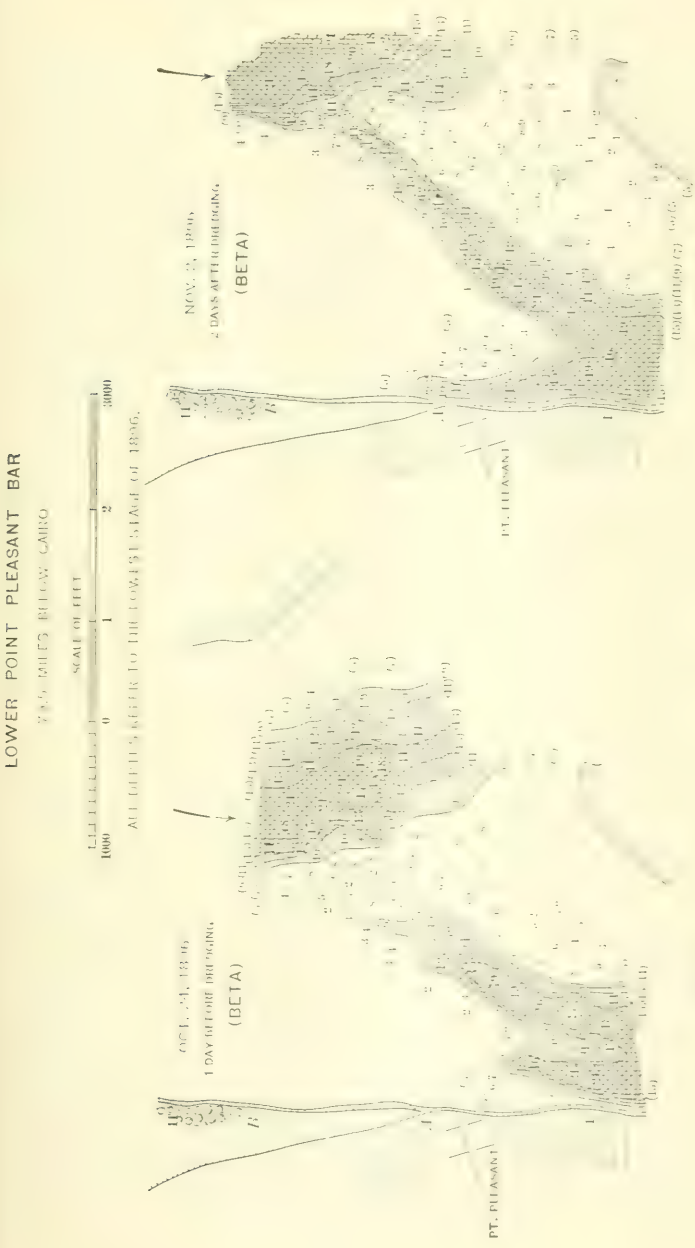

LOWER POINT PLEASANT BAR
SCALE OF FEET
(BETA)
PT. PLEASANT
(BETA)
PT. PLEASANT

Fig. 13.

Fig. 14.

TABLE No. 10.—Summary of Dredging Operations, Mississippi
of

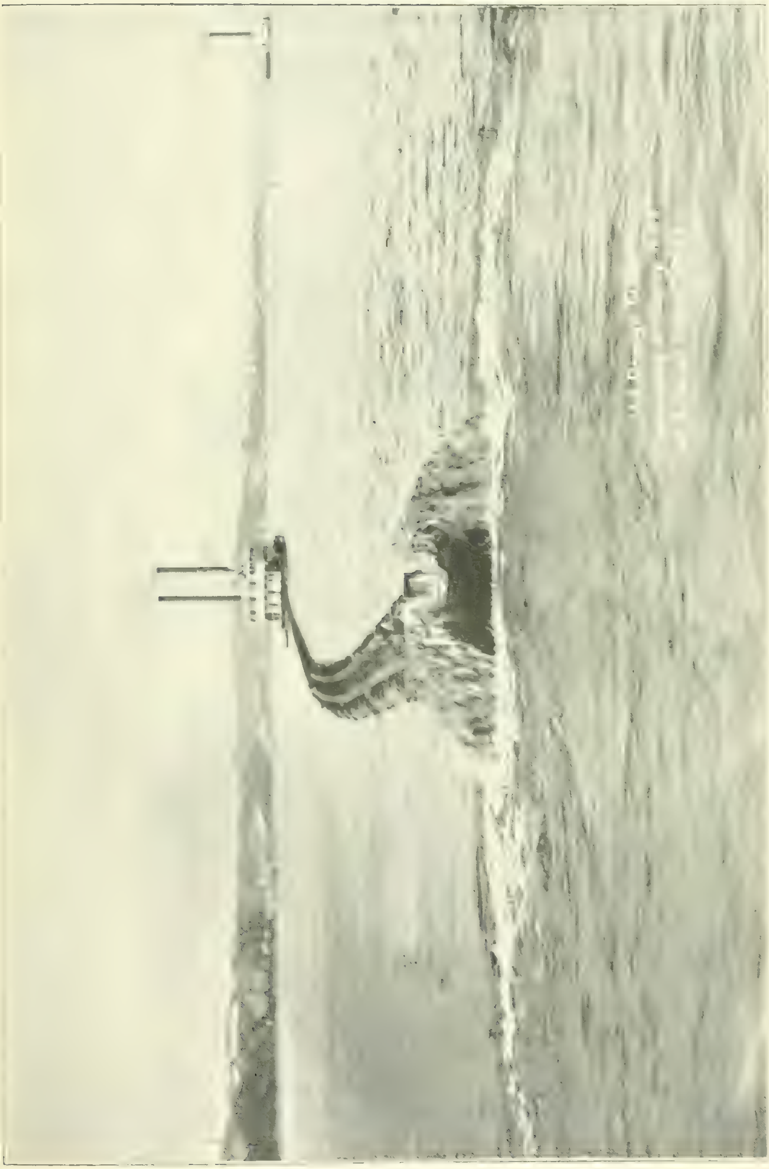
Place and date.	Name of dredge.	Cuts. Number.	Cuts. Total length. Feet.	Cuts. Time required. Hours.	Cuts. Rate, ft. per hour.	Average depth in feet. Suction.	Average depth in feet. Ahead of dredge.	Average depth in feet. Astern of dredge.	Average depth in feet. Material removed.
[illegible] August 31st to September 24th, 1896....	*Beta.*	10	5 945	121.0	49.1	15.3	9.2	13.4	4.2
[illegible] September 25th to 30th, 1896..........	*Beta.*	3	2 265	66.0	34.5	15.6	9.5	13.5	4.0
[illegible] September 30th to October 9th, 1896....	*Beta.*	9	5 660	63.7	89.0	15.0	9.4	13.5	4.1
Hathaways Lower Crossing (102 L). October 9th to 15th, 1896	*Beta.*	16	12 175	87.0	140.0	20.5	14.8	18.7	3.8
Cherokee Crossing (89 R), second time. [illegible]	*Beta.*	11	6 435	78.2	82.3	17.0	10.8	15.8	5.0
Lower Point Pleasant, Mo. (79.5 R). October 25th to November 3d, 1896	*Beta.*	13	9 415	109.2	86.2	16.0	7.7	13.5	5.8
Cherokee Crossing (89 R), third time. [illegible]	*Beta.*	10	8 980	175.0	51.3	17.3	10.7	16.1	5.4
Cherokee Crossing (89 R), first time. September 5th to 28th, 1896............	*Alpha.*	6	4 610	294.0	15.7	12.0	7.8	10.9	3.0
Upper Point Pleasant, Mo. (79 R). September 29th to October 23d, 1896....	*Alpha*	4	6 370	306.7	20.8	13.0	9.3	11.7	2.4
Compromise Bar (77 L). October 24th to November 15th, 1896....	*Alpha.*	7	5 405	257.7	21.0	12.7	9.9	11.6	1.7

the cut. Or, on the other hand, if the cut has been so located as to meet the requirements of the current, a channel will be scoured out and an amount of material removed which would probably exceed the capacity of the dredge many times. It is by no means certain that the channel will always follow the line of the cuts. It sometimes happens that after some days of diligent dredging, a channel develops without assistance on a line quite remote from the dredged cuts.

Whatever is done during one low-water season is generally obliterated by the succeeding high water. The successful location of a cut during one low-water season does not necessarily imply that a similar location during the following season will give like results. In fact, a new problem is presented by each bar every season.

In a stream of this character it is, of course, extremely difficult to determine in a particular case just what share of the channel should

PLATE XLIII.
PAPERS AM. SOC. C. E.
JUNE, 1898.
OCKERSON ON DREDGES AND DREDGING.



River Between Cairo and Memphis. During Low Water Season 1896.

Heads in feet.		Revolutions. Ave. per min.		Distribution of time by hours.										
Suction.	Delivery.	Pumps.	Cutters.	Steam pressure.	Placing plant.	Dredging.	Changing cuts.	Repairs.	Passing boats.	Making up tow.	Towing.	Sundays and holidays.	Total time.	Total cost.
14.4	19.1	129	15	161	53.5	121.0	25.2	12.7	1.0	39.0	104.0	19.0	375.5	$6 122.51*
15.2	21.6	134	16	165	16.5	66.0	5.0	14.5	......	6.5	25.5	......	134.0	1 793.42
15.2	20.8	135	15	167	11.0	63.7	7.7	6.5	......	20.5	97.5	10.0	217.0	2 379.05
15.0	23.4	134	16	160	23.0	87.0	......	.....	......	5.5	28.0	......	143.5	1 671.59
14.5	24.5	135	16	163	9.5	78.2	......	9.7	......	18.0	24.0	......	139.5	1 742.58
15.2	25.6	136	16	162	13.0	109.2	......	62.7	......	7.0	24.0	24.0	240.0	2 315.51
13.6	25.9	137	14	164	14.0	175.0	......	24.0	......	24.0	......	3.0	240.0	3 008.93
5.2	16.8	124	Water-jet.	138	32.0	294.0	11.0	143.0	......	24.0	8.0	64.0	576.0	3 309.51+
6.3	19.5	118		141	14.2	306.7	15.7	79.2	......	......	9.0	70.0	495.0	3 456.16+
5.8	20.4	120		139	13.5	257.7	8.25	93.0	......	37.5	10.5	59.5	480.0	3 473.68+

* Capsizing of pipes delayed the work 224½ hours.
\+ 114¾ hours. }
78 hours. } Repairs due to breaking of runner of main pump.
91 hours. }

be credited to the dredging and what proportion to natural causes. Enough bars have been dredged, however, to fully demonstrate that the dredging at least induces the scour which eventually results in a wide and deep channel.

During the low-water season of 1896 there were eight bars between Cairo and Memphis which obstructed navigation. Channels were successfully opened through all of them except one, which was abandoned on account of an extraordinary change in the river which shifted the channel from the right to the left bank. One of the most interesting of these bars is shown in Figs. 13, 14, 15 and 16. Fig. 13 shows that one day before dredging there was barely a 7-ft. channel between the upper and lower pools. The broken lines enclose the area where the dredging was done. The controlling depth in this area was 6 ft.

Fig. 14, two days after dredging was completed, shows the two pools connected by a good 9-ft. channel with 11 ft. of water for the greater part of the length. Fig. 15 shows the conditions seventeen days after dredging. The cut has deepened and an 11-ft. channel from pool to pool, and 15 ft. nearly all of the way through, is found. Fig. 16 shows, in a forcible way, the magnitude of the natural movement of sand bars. Thirty-three days after dredging, the channel was found to be about 800 ft. below the cut opened by the dredge. The site of the dredged channel, which in Fig. 15, only sixteen days earlier, showed 15 ft. of water, now has barely 4 ft. The 9-ft. channel, however, still remains, although it has drifted down stream. This change occurred during a rapid rise at the end of the low-water season of 1896.

A summary of the dredging operations during the season of 1896 is given in Table No. 10.

The following table shows the cost of operating the two dredges during the low-water season of 1896:

Labor	$16 237.06
Subsistence	3 386.67
Fuel	8 051.73
Oil	242.39
Miscellaneous engineers' supplies	176.43
Mates' supplies	94.73
Repairs	855.41
Lighting supplies	114.58
Miscellaneous	113.94
Total	$29 272.94

This only covers the cost of operation at the several bars, and does not include the cost of repairs prior to the beginning of work, the expenses incurred while waiting developments in the stage of river to see whether further work was necessary or not, nor the cost of moving to winter quarters. These items amount to $12 953.23, giving a total for the season of $42 226.17.

During the low-water season of 1897 four dredges were at work on the bars below Cairo. The cost of operating them is given in the table on page 519.

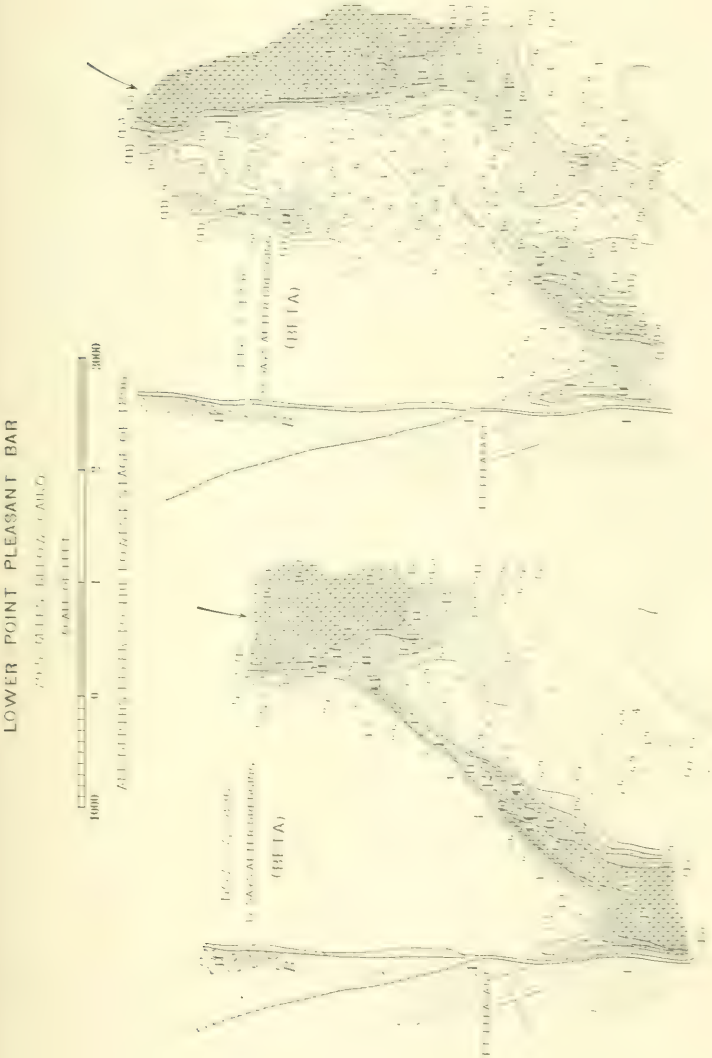


Fig. 15

Fig. 16

COST OF OPERATING DREDGES DURING LOW-WATER SEASON OF 1897.

Labor	$26 704.53
Fuel	13 474.23
Subsistence	7 586.99
Lubricants	970.79
Lighting supplies	215.05
Miscellaneous supplies	1 527.79
Office supplies	145.10
Hire of tow boats for moving plant	10 115.50
Preparation and moving into field	3 269.44
Moving to winter quarters at end of season	4 637.03
Cost of inspection and supply boat	10 554.75
Cost of operating two survey boats	8 674.10
Total	$87 875.30

Fig. 17 shows a conspicuous sand bar at Lazelles, some 82 miles below Cairo. When the survey of this bar was made, just before dredging, the distance from the 9-ft. depth in the upper pool to the same depth in the lower pool was 1 400 ft. At that time the amount of excavation required to make a 9-ft. channel 250 ft. wide at the datum stage was 61 000 cu. yds. The dredge *Gamma* was at work on this bar ten days, and the actual dredging time was one hundred and seventy-eight hours. Taking the capacity of the dredge as 800 cu. yds. per hour gives the total amount dredged as 142 400 cu. yds. Thirty-one days after the dredging was completed there was a 10-ft. channel clear through the reef, except for a distance of about 225 ft. where the depth was only 8 ft. (see Fig. 18). Seventy-six days after the dredging was completed the channel was found to be 350 ft. wide at the narrowest point, over 1 000 ft. wide for more than nine-tenths of the length, and with a least depth of 9 ft., and an average depth of 11 ft. (see Fig. 19).

At Hathaways, about 104 miles below Cairo, the surveys showed a very ragged 7-ft. channel before dredging. One and a half days after dredging, a narrow 9-ft. channel connected the upper and lower pools, and a clear 8-ft. channel 350 ft. wide extended across the bar. Thirty days later the channel was found to be 12 ft. deep and 800 ft. wide with the exception of a narrow shoal at the lower end.

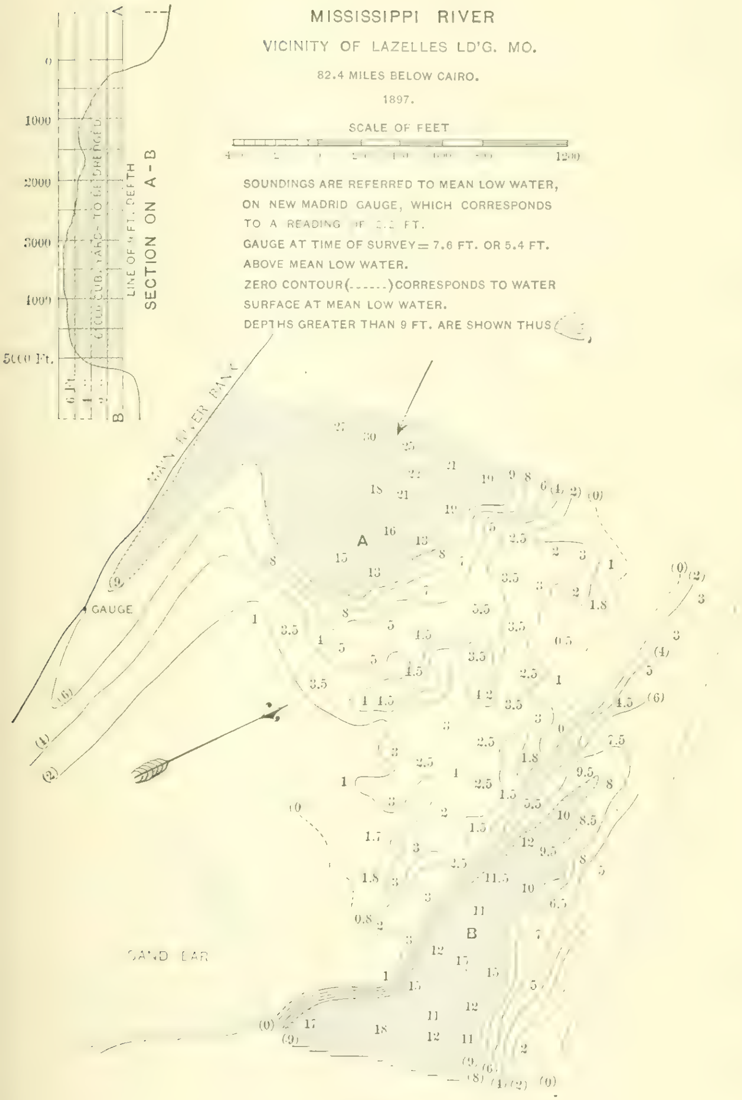

MISSISSIPPI RIVER
VICINITY OF LAZELLES LD'G. MO.
82.4 MILES BELOW CAIRO.
1897.
SCALE OF FEET
SOUNDINGS ARE REFERRED TO MEAN LOW WATER,
ON NEW MADRID GAUGE, WHICH CORRESPONDS
GAUGE AT TIME OF SURVEY = 7.6 FT. OR 5.4 FT.
ABOVE MEAN LOW WATER.
ZERO CONTOUR(------)CORRESPONDS TO WATER
SURFACE AT MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
SECTION ON A - B
GAUGE
A
B
SAND BAR

FIG. 17.

At Beef Island, 211 miles below Cairo, the channel was deepened 2 ft. in one hundred and six hours of actual dredging by the dredge *Delta*, in spite of the presence of many sunken logs and snags.

At President's Island, just below Memphis, the dredge *Delta* was employed in opening a channel through a reef which was causing much trouble to navigation. In the bed of the river in this locality there is a considerable quantity of conglomerate rock and lignite to be found. The mechanical agitators of the *Delta* were therefore put to a very severe test. One solid piece $3\frac{1}{2}$ x $4\frac{1}{2}$ ft. was caught in the cutters (see Figs. 1 and 2, Plate XLII). It had to be broken into small pieces to dispose of it. When work began there was barely a 5-ft. navigable channel across this bar. After fifteen days' work, during which period much time was lost in clearing the cutters of debris, leaving only eighty hours' actual dredging time, a 9-ft. channel 100 ft. wide and 1 300 ft. long was completed. This channel remained intact throughout the season, although a wider channel broke through the bar lower down.

During the low-water season of 1897 channels were dredged through fifteen bars lying between Cairo and Graves Bayou, 248 miles below. A view of a dredge at work, showing the pipe line and the discharge against the baffle plate, is shown in Plate XLIII.

There were bars that obstructed navigation below Graves Bayou, as far down as Arkansas City, 435 miles below Cairo, and one bar even below Natchez, 710 miles below Cairo. The dredging plant, however, was too small to cover more than the 250 miles from Cairo southward. The total number of obstructing bars below Cairo during the low-water season of 1897, where the navigable depth was less than 8 ft., probably did not exceed twenty-five. Estimating each of these bars at 2 000 ft. in length, the result is an aggregate of about $9\frac{1}{2}$ miles of bad navigation in a total length of 1 060 miles of river from Cairo to the Gulf of Mexico, or less than 1% of the length.

Estimating the average depth of cut required as 4 ft. and the width at 250 ft., gives the total amount of material to move as 1 852 000 cu. yds. To remove this amount in ten days would require eight dredges running the full time without interruption. Table No. 8 shows that only about half of the time is utilized in actual dredging.

These bars do not all develop at the same time, and they do not necessarily develop at the same places during successive seasons.

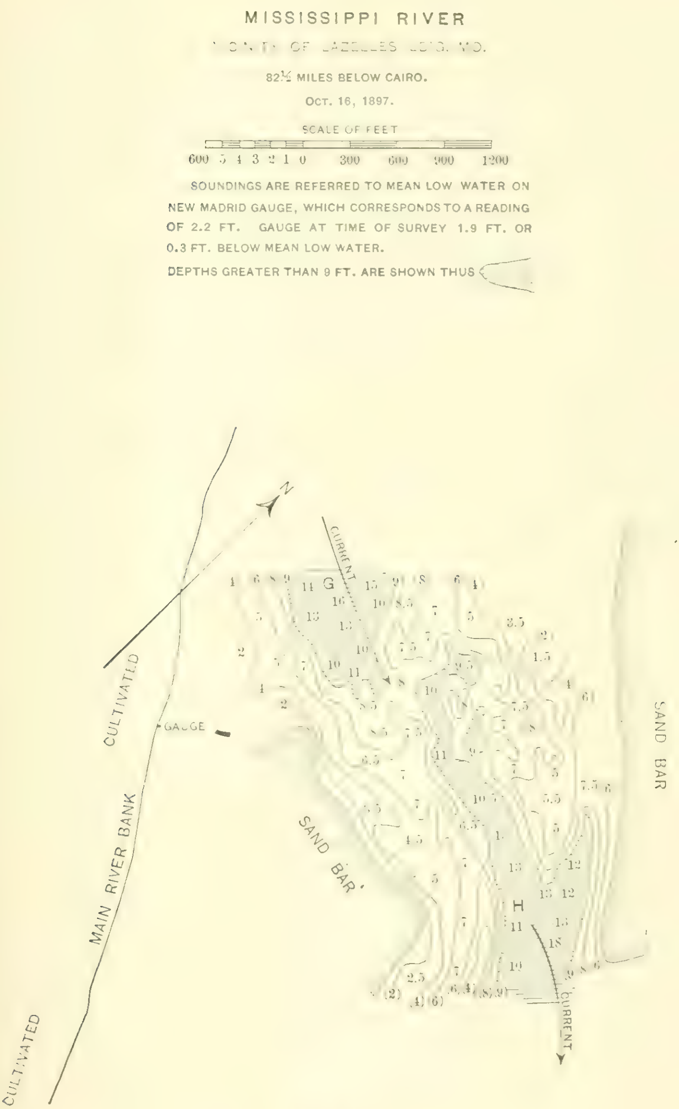

MISSISSIPPI RIVER
82½ MILES BELOW CAIRO.
OCT. 16, 1897.
SCALE OF FEET
600 5 4 3 2 1 0 300 600 900 1200
SOUNDINGS ARE REFERRED TO MEAN LOW WATER ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.2 FT. GAUGE AT TIME OF SURVEY 1.9 FT. OR 0.3 FT. BELOW MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
N
CURRENT
G
H
GAUGE
CULTIVATED
MAIN RIVER BANK
CULTIVATED
SAND BAR
SAND BAR
CURRENT

FIG. 18.

There are, however, certain general localities that have long been recognized by steamboatmen as liable to develop bad navigation during any low-water season. Now and then, even these places pass through a season without developing any serious obstructions, and at such times dredging below Cairo will be unnecessary. Seasons having good navigation throughout are, however, not frequent.

Dredging operations will, in the main, be confined to the same general localities from year to year, although wrecks, snags and bank erosions may occasionally develop new bars that will require dredging.

The season of 1895 was one of extraordinary low water. At one time during that season there were thirty-seven crossings between St. Louis and Cairo, where the water was 6 ft. or less in depth. During ordinary seasons about as much dredging would be required between St. Louis and Cairo to secure a 9-ft. channel 250 ft. wide as is required below Cairo for a similar channel. The supply of water above Cairo at extreme low stages is too limited to readily admit of 9-ft. channel depths. The requirements of river traffic on this reach would be satisfied fairly well by 7 or 8 ft., and it is doubtful whether these depths can be materially exceeded by means of dredges or other temporary devices at an expense justified by the value of the results obtained.

It is evident from the foregoing that the problem of keeping navigation open in the Mississippi River during the low-water season is very far from being a simple one. The magnitude of the plant described is sufficient evidence that strenuous efforts are being made to "improve and give safety and ease to the navigation" of the river, and the experience gained thus far justifies the belief that these efforts will be successful.

The writer is indebted to General John M. Wilson, Chief of Engineers, U. S. A., M. Am. Soc. C. E., to Major Thos. H. Handbury, Captain H. E. Waterman and other officers of the Engineer Corps, for free access to records and drawings. He is also indebted to his associates, engaged in the improvement of the river, for valuable assistance in the preparation of drawings and tabulated data.

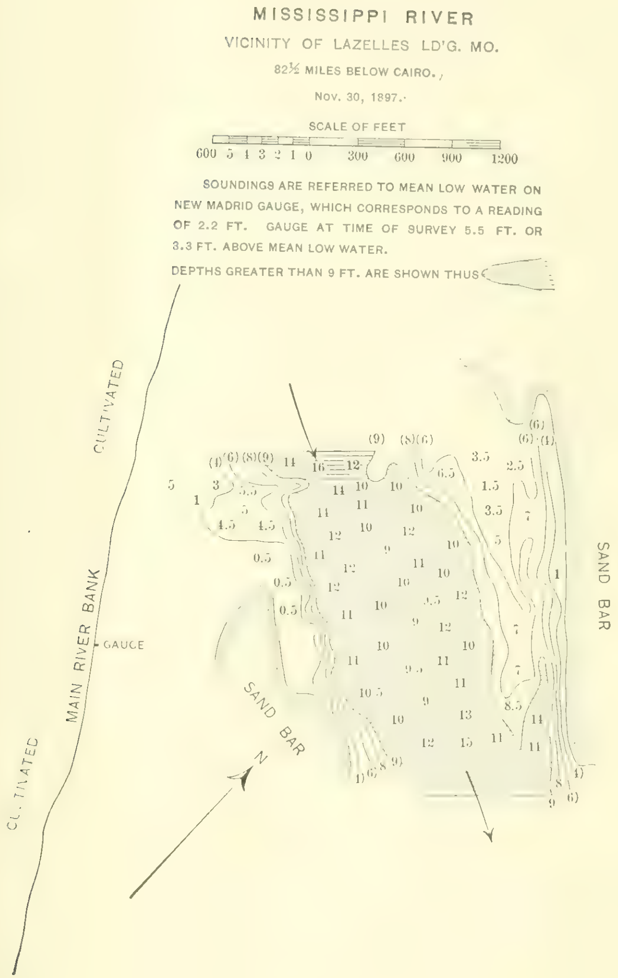

MISSISSIPPI RIVER
VICINITY OF LAZELLES LD'G. MO.
82½ MILES BELOW CAIRO.
Nov. 30, 1897.
SCALE OF FEET
600 5 4 3 2 1 0 300 600 900 1200
SOUNDINGS ARE REFERRED TO MEAN LOW WATER ON NEW MADRID GAUGE, WHICH CORRESPONDS TO A READING OF 2.2 FT. GAUGE AT TIME OF SURVEY 5.5 FT. OR 3.3 FT. ABOVE MEAN LOW WATER.
DEPTHS GREATER THAN 9 FT. ARE SHOWN THUS
CULTIVATED
MAIN RIVER BANK
GAUGE
SAND BAR
SAND BAR
N

FIG. 19.

1898. JUNE, No. 6.

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PAPERS.

RESERVOIR SYSTEM OF THE GREAT LAKES OF THE ST. LAWRENCE BASIN;

Its Relation to the Problem of Improving the Navigation of these Bodies of Water and of Their Connecting Channels.

By Hiram M. Chittenden.*

WITH A MATHEMATICAL ANALYSIS OF THE INFLUENCE OF RESERVOIRS UPON STREAM FLOW.

By James A. Seddon, C. E.†

The immense system of inland navigation, known as the Great Lakes of the St. Lawrence basin, possesses many features in common with the navigation of the high seas. These bodies of water are of great magnitude. The vessels which sail upon them rival in size sea-going craft, and are built to withstand storms which approach in fury those of the ocean itself. Light-houses and fog signals define the position of dangerous coasts, and breakwaters are required to protect the open roadsteads. The lake ports are essentially maritime ports, and the connecting channels resemble maritime straits and canals more than they do ordinary rivers. The commerce of the lakes has

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now reached a magnitude which makes a respectable comparison with the ocean commerce of various parts of the world. Several of the lake ports do a business exceeded by but few seaports, and the commerce which passes the Sault Ste. Marie Canal and the St. Clair and Detroit rivers surpasses in volume that of the world's greatest maritime canal. Similarity of conditions has given rise to similarity of problems, such as the protection of ports, the deepening of harbors, and the enlargement of connecting channels, which are common objects in the interests of navigation, alike on the lakes and on the ocean.

There are, however, two important physical characteristics which create a notable difference between the conditions of ocean and lake navigation. One of these is the existence of diurnal tides on the ocean which presents problems of a distinct character not met with except in tidal waters. The other belongs to the lakes, and relates to their condition as bodies of fresh water with a perennial overflow. This characteristic not only gives rise to problems new and distinct in themselves, but it affords new solutions for old problems pertaining to maritime navigation in general. It is to a consideration of this particular feature of the navigation of the Great Lakes that the present paper is devoted.

Notwithstanding the vast magnitude of these lakes and their resemblance in many particulars to tidal waters, they are, after all, only parts of an immense river system which drains a large area of country. They are subject to the variable conditions of water supply characteristic of all streams. The connecting channels and the final outlet carry a continuous current, always in one direction, unlike maritime straits and canals through which the water may flow in either direction depending upon the state of the tide or of the wind. But while the lakes and their connecting channels thus constitute a great river system, that system differs from nearly all others in this particular—that the flow in the outlets is practically exempt from those variations and irregularities which obtain upon nearly all other streams.

The explanation of this most important characteristic is to be found, of course, in the controlling action of these lakes as great reservoirs interposed in the course of the stream. The storage represented by a few inches rise and fall in the lake levels is enormous, and when withheld during the flood season and released in the dry season, it gives to the outlets a regimen of flow which is radically dif-

ferent from that of an ordinary river. This influence is not annual merely, but cyclic as well, and a series of what may be called wet years causes a general rise of mean level, the storage of which maintains the flow in the outlets during the ensuing years of less than average precipitation. The balance of forces which Nature has here produced in the course of long ages is one of the most marvelous features of these lakes; and a careful contemplation of it cannot fail to convince one that an almost perfect compromise has been reached between the conflicting oscillations of lake level and outlet discharge. It is difficult to see how either could be brought nearer to absolute uniformity without a resulting departure in the other which would more than offset the gain.

Perfect as this natural condition is, however, it yet does not satisfy the ambition of those who make commercial use of it. There are many who consider the oscillation in lake levels as an evil which ought not to be suffered to continue. The ship-owner, who loads his boat to the full limit of harbor depth, complains when the waters subside and compel him to load to a lighter draft. The periodic oscillation in particular, which every now and then results in a continuous subsidence of mean level through a series of years, is very damaging to commercial interests and naturally causes a good deal of apprehension. Lake carriers are led, on account of it, to clamor for a corrective, which, if applied, might entail greater evils than those of a falling lake level, for they apparently forget that it is this very subsidence of level which keeps the water in the connecting channels from subsiding in a much greater degree.

Various causes have contributed to give this subject unusual prominence in recent years. A period of what might be called dry years culminated in 1895, in the lowest mean level in Lakes Michigan, Huron and Erie that has ever been experienced since the commerce of the lakes has been a matter of great moment. The artificial enlargement of the natural cross-sections of some of the connecting channels, for the improvement of navigation, has been thought by many to have contributed to this result. Simultaneously with the continuance of this unfavorable condition were the commencement and vigorous prosecution of a project which contemplates the permanent withdrawal of a large amount of water from Lake Michigan, the possible maximum of the diversion being 10 000 cu. ft. per second. All of these coincident

causes of alarm created no little uneasiness among commercial interests, and gave rise to a great deal of earnest consideration as to ways and means of averting the threatened dangers. Several public discussions and a considerable amount of professional literature on the subject were the first result. The matter finally came up in Congress in the shape of several reports which were made the basis of a bill* to provide for a thorough investigation of the whole question. In the meanwhile Nature commenced removing the immediate cause of apprehension, for the mean lake levels have been rising since 1895, and very likely will continue to rise for some time to come. As a natural result, the urgent clamor for action, which was so prominent a few years ago, subsided as the lake levels rose, and will probably not assume troublesome proportions again until a new period of low water arrives. The problem, nevertheless, is before us for solution, and the respite afforded by the natural relief now being enjoyed should not be permitted to go unimproved, but should be utilized to the utmost to bring about a well-digested plan of action before the hour of necessity again arrives.

This problem, as developed by the general discussions already referred to, aims to secure three objects:

(1) A control of the lake levels by which their annual and periodic oscillations may be, if not altogether eliminated, still materially reduced in range. This purpose is expressed in the following extract from the bill proposed to Congress in the winter of 1895-6, which directed, among other things, an inquiry and an investigation as to whether

"it is practicable to control the waters of the Great Lakes and maintain them at substantially a uniform level at all seasons of the year, by a dam or dams or other works placed in Niagara River at the outlet of Lake Erie, and by a system of wing dams or other structures placed in the Detroit River and the St. Clair River, and the Sault Ste. Marie or St. Marys River, at or near the respective outlets of Lakes Saint Clair, Huron and Superior."

The words "substantially a uniform level" are not specifically interpreted, but a limit of oscillation of 6 ins. has been urged as a practicable one, at least for Lake Erie.

(2) A permanent elevation of the lake levels by means of contracting works in the outlets. This, it is urged, would be the sim-

* This particular bill did not become a law.

plest, most effective and most economical method of securing a permanent increase of depth in the ports and connecting channels of the lakes.

(3) Some means of counteracting the tendency to a permanent lowering of lake levels due to an artifical enlargement of natural cross-sections of the connecting channels, or to the permanent diversion of any portion of the overflow to another water-shed.

It may be accepted as a preliminary condition, to which there can be no qualification, that any alteration of natural conditions in the Great Lakes, which shall result in a material diminution of flow in the outlets during any portion of the navigation season, cannot be considered for a moment. It therefore becomes a matter of the first importance to determine what effect the attainment of any of the above objects will have on the flow in these outlets.

Inasmuch as the navigation system of the Great Lakes, as already pointed out, is virtually a great river regulated by immense reservoirs distributed along its course, it is manifest that the hydraulic features of the problem are inseparably bound up with the laws of reservoir action in the regulation of stream flow.

Fundamental Propositions Pertaining to the Action of Reservoirs in the Regulation of Stream-Flow.

I.—*The effect of a natural reservoir upon a stream passing through it is to reduce variations and irregularities of flow and to give the stream a more uniform discharge upon leaving the reservoir than it had before entering.*

The explanation of this common phenomenon is obvious. When the inflow into a reservoir increases, the outflow will not generally increase at the same rate, for a portion of the inflow will be absorbed in storage due to a rise in the reservoir level. In like manner, when the inflow is decreasing, the outflow will not generally decrease at the same rate, for it will be reinforced by the storage which flows out as the reservoir falls. So that, while the total outflow may equal the total inflow, it will not generally show the same extremes of variation. The effect is analogous to that of a balance wheel upon the motion of machinery; the wheel being virtually a reservoir of force, in which irregular and sudden impulses are stored, and from which the stored force is paid out to sustain motion whenever there are sudden or irregular cessations in the application of the moving force.

To give mathematical expression to this proposition:

Let S represent supply in the most general sense of that term, positive and negative, including inflow from other reservoirs, rainfall, run-off, evaporation, and all other sources of supply or loss except the discharge of the outlet. It is expressed in second-feet and may have negative values;

Let Q represent the discharge of the outlet in second-feet;

Let h represent the variable height in feet of the reservoir surface above any datum, and also of the outlet immediately at the reservoir;

Let t represent the time in seconds from an assumed origin to any value of h;

Let A represent the area of the reservoir in square feet.

Then, at any instant, the rate of change in h will be represented by the following equation:

$$(S - Q)\, d\, t = A\, d\, h \qquad (1)$$

The equation for the discharge of the outlet is

$$Q = f\,(h) \qquad (2)$$

in which $f\,(h)$ always increases and diminishes with h, its particular form, in any given case, depending upon the character of the outlet.

From equation (1) it is seen that whenever h is increasing, $d\,h$ is positive, and $Q < S$; and that whenever it is diminishing $d\,h$ is negative and $Q > S$; also, that when h and Q are at maximum or minimum, $d\,h$ is zero and $Q = S$.

In any cycle of changes, the maximum value of S precedes and is greater than the maximum value of Q. For if the curves of S and Q were both plotted to the same time scale, and if both maxima were equal and simultaneous, the two curves would be tangent to each other at that point where $Q = S$. But it has been seen that when Q is approaching its maximum it is less than S, and that after leaving its maximum it is greater than S. The two curves thus intersect at the point where $Q = S$, and therefore cannot be tangent; and since, at this point, Q is at a maximum and S is falling, S must have already passed a maximum greater than that of Q. In like manner, it will be seen that the minimum of Q follows, and is greater (algebraically) than that of S. The limits of oscillation in Q, therefore, lie within those of S.

From equation (1) it will also be seen that, other things being equal, the moderating effect of a reservoir upon stream flow varies directly with its area.

In the general proposition above stated the word "natural" was used by way of qualification, because in the practical uses to which artificial reservoirs are often put, the proposition is not strictly true. In some reservoirs the outlets are entirely closed during certain periods, and the water is all released at other periods, so that, in such cases, the natural fluctuations in the flow of the stream may actually be increased, or at least be radically altered in their periods.

Even in natural reservoirs there are apparent, if not real, exceptions. If a reservoir be very small in comparison to the volume of water flowing through it, but little larger in fact than the channel of the stream itself, the rise in the reservoir will follow so closely upon that in the stream that the two may appear to be simultaneous and equal. This is but one illustration of what may be stated as a general rule, that every stream is virtually a series of small reservoirs and connecting channels; and that all streamflow is more or less subject to the influence of reservoirs.

Reservoirs in Series.

As a corollary to the foregoing proposition, it follows that, if several reservoirs succeed each other in a descending series, so that a supply to any reservoir must flow successively through all below, the fluctuation of level in the different reservoirs, due to a variable supply to an upper reservoir, will diminish with a descent of the series and will approach zero as a limit.

It has been seen that in any reservoir the variation of Q is less than that of S; but, neglecting local supply, Q of one reservoir becomes S of that next below. Calling this S', and the discharge of the lower outlet, Q', there results:

$$Variation\ Q' < Variation\ S' < Variation\ S, \text{ etc.};$$

but $Variation\ h = \frac{Variation\ Q}{c}$ (see notation just below), in which c, as a rule in natural reservoirs, will increase with a descent of the series. The change in each term of the above ratio is therefore such as to diminish the value of *Variation* h*.

* The term "variation," as used in this paper, is not to be taken in its mathematical signification, but simply as denoting the maximum fluctuation in S and Q. It corresponds to the term "oscillation," as applied to h.

There are possible exceptions to this rule. If it should happen that a particular reservoir were of very small area, with an engorged outlet, in which the value of c were actually smaller than that in the outlet next above, the value of $\frac{\text{Variation } Q}{c}$ might be greater than for the next preceding reservoir. Examples of this are common in the flow of nearly all streams. When the channel of a river is engorged from any cause, the value of c is less at this point than either above or below. If the river valley above be considered as a very small reservoir, we shall have a true example of the above exception. It is known, as a matter of fact, that the rise of rivers in times of flood varies materially at different points of their course owing to this cause. But while this exception is true of such extreme cases as have been here cited, it would not be true of any ordinary system of reservoirs.

For a more specific study of these relations:

Let A denote the area of the reservoir in square miles.

Let h denote the height in feet of the surface of any reservoir above the mean level for the cycle. For the first reservoir considered, it will be written h_1; for the next reservoir the resulting oscillation will be h_2, and so on to h_n.

Let c denote the variation of Q in second-feet for a change of 1 ft. in h, $f(h)$ being assumed to be a right line.*

Let P denote the coefficient of reduction of S, or the ratio $\frac{Q \text{ (maximum)}}{S \text{ (maximum)}}$.

Let p denote the coefficient of reduction of h, or the ratio of h for one reservoir divided by h for that next above, or $\frac{h_n \text{ (maximum)}}{h_{n-1} \text{ (maximum)}}$.

Let R denote the interval which elapses between the maximum or minimum of S in any reservoir, and the resulting maximum or minimum of Q. R is expressed in degrees of arc; a full circle, or 360°, representing a complete cycle, taken in this case as one year.

Let the variations of supply be represented by the equation $S = a \sin t$, in which a is the extreme variation in second-feet above or below the mean, and t is degrees of arc, of which 360 make a complete cycle.

* In streams of considerable magnitude and small variation in discharge, the assumption that $f(h)$ is a right line for slight changes in h, involves so small an error as not to impair in the least degree the validity of the mathematical processes in which $f(h)$ enters.

Then the following equations may be written:*

$$\text{Sin. } R_n = \frac{1}{\sqrt{1 + \left(\frac{0.18 c_n}{A_n}\right)^2}} \qquad (3)$$

$$P_n = \frac{1}{\sqrt{\left(\frac{A_n}{0.18 c_n}\right)^2 + 1}} \qquad (4)$$

$$p_n = \frac{0.18 c_{n-1}}{\sqrt{A^2_n + (0.18 c_n)^2}} \qquad (5)$$

From these equations it is seen that, for a given cycle, R is a function of area and outlet alone, and independent of S; P also is a function of area and outlet alone; and p is a function of area and of inlet and outlet.

It will also be seen that R decreases as A increases, or as c of the outlet decreases; P increases as A decreases, or as c of the outlet increases; while p increases as A and c of the outlet decrease, and as c of the inlet increases. In each of the above cases the converse statement is true.

Inasmuch as the conditions which control the oscillation of level in natural reservoirs are seldom, if ever, such as to bring the crests of transmitted waves from upper reservoirs, and those due to local supply, simultaneously together, wave interference, which to some extent neutralizes the combined effect, is the result. In other words, the maximum oscillation of level in any reservoir of a series is never the sum of the maximum of local and transmitted oscillations, for these are never exactly superimposed; and they may even so occur that the hollow of the transmitted wave comes upon the crest of the local wave, and actually cause a smaller oscillation than if there were no transmitted wave.

As a general proposition it may be said that, in a series of natural reservoirs, such as has been described, each with its independent local supply, the oscillation of levels will be mainly controlled by the transmitted supply when the reservoir area is relatively small, and by the local supply when its area is relatively large.

For the better illustration of the principles above enunciated, let the following example be considered. Let there be five reservoirs,

* See mathematical analysis hereto appended.

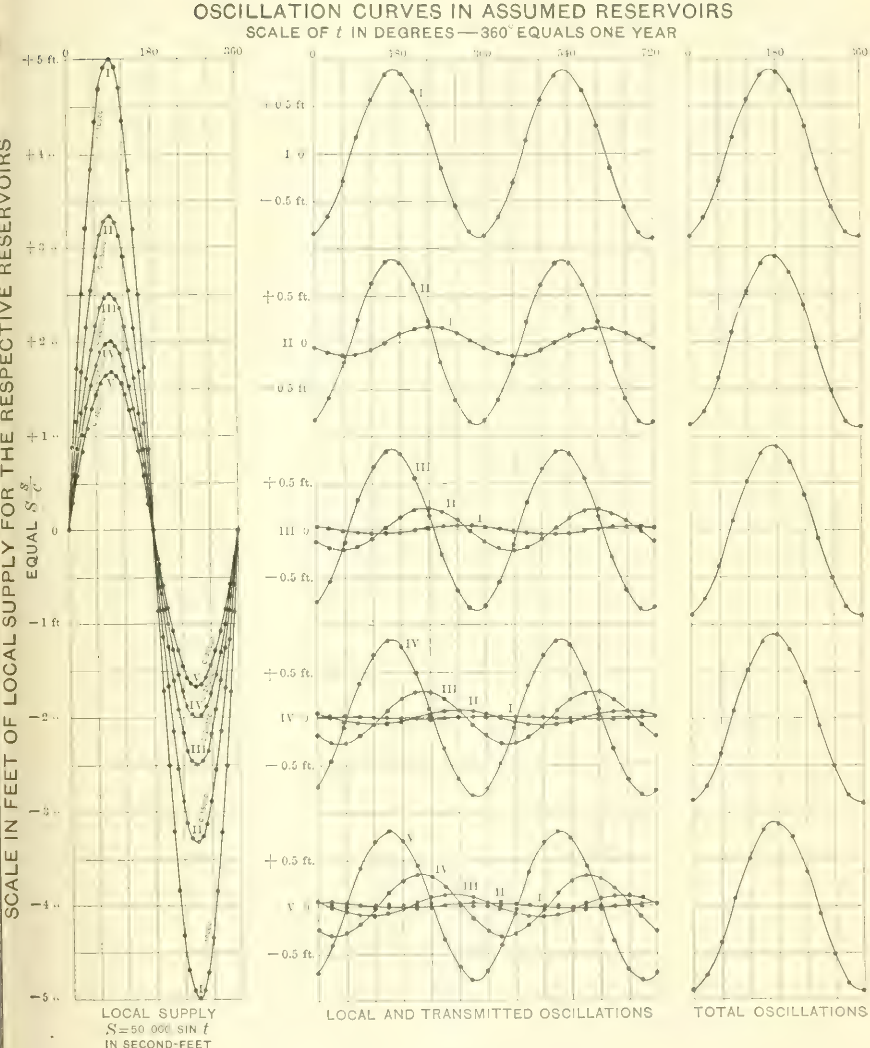
OSCILLATION CURVES IN ASSUMED RESERVOIRS
SCALE OF t IN DEGREES—360° EQUALS ONE YEAR
SCALE IN FEET OF LOCAL SUPPLY FOR THE RESPECTIVE RESERVOIRS
EQUAL $S\frac{S}{C}$
LOCAL SUPPLY
S=50 000 SIN t
IN SECOND-FEET
LOCAL AND TRANSMITTED OSCILLATIONS
TOTAL OSCILLATIONS

FIG. 1.

and assume the area of each to be 10 000 square miles, the range of variation in S to be 100 000 second-feet, and to be the same for each of the reservoirs; and the values of c, beginning with the first reservoir, to be 10 000, 15 000, 20 000, 25 000 and 30 000 second-feet. The values of R, P and p, as computed from equations (3), (4) and (5), may then be written as follows:

Reservoir.	R.	P.	p.
No. 1	81 days.	0.177	.....
No. 2	76 "	0.261	0.174
No. 3	71 "	0.339	0.254
No. 4	67 "	0.410	0.328
No. 5	62 "	0.475	0.396

The accompanying oscillation curves (Fig. 1) show for each reservoir the element of total oscillations which comes from local supply, and that from each of the reservoirs above, and enables one to judge at once by the eye what proportion of the total oscillation in any case is due to local supply, and what to transmitted supply. In the example above given, in which the reservoirs are relatively large, the oscillation is mainly due to local supply. For instance, in the third reservoir, the maximum oscillation, due to local supply, is 95½% as great as the actual oscillation, and occurs 12 days earlier. The maximum transmitted oscillation is 25½% as great as the actual, and occurs 68 days later.

To show what would be the effect upon a comparatively small reservoir, suppose that in the above series, in place of the third unit, there is introduced a reservoir of the following size and characteristics: Area, 400 square miles; range of variation in S, 4 000 second-feet; c of outlet, 20 000 second-feet. It will then be found that the maximum of oscillation, due to local supply, is 30% as great as the actual, and occurs 67 days earlier; while the maximum of oscillation, due to transmitted supply, is 94% as great as the actual, and occurs 20 days later. In this reservoir, therefore, the oscillation of level would be mainly controlled by the supply received from above.

II.—*Any reduction of the normal oscillation of level in a natural reservoir will increase the variation of flow in the outlet; and any reduction of the normal variation of flow in the outlet will increase the oscillation of level in the reservoir.*

From equation (1) it will be seen that, if the oscillation of level of a reservoir is to be diminished, $(S - Q)\,d\,t$ must be diminished, and consequently the values of $(S - Q)$ must be diminished; but since the variation of S from its mean is greater than that of Q, and since the variation of S is here assumed to remain unchanged, Q must be brought nearer to S, and its own variation must be increased.

The truth of the above proposition is readily apparent without mathematical demonstration. If a reservoir be prevented from rising by any amount, the water represented by the storage under normal conditions must be run out, and the flow in the outlet must be correspondingly increased. With a falling reservoir exactly the opposite process applies, and the flow of the outlet must be diminished by any restriction of the normal amount of fall. Finally, if the oscillation of level be eliminated altogether, Q will become equal to S—the condition of an unreservoired stream.

The importance of this proposition arises from its relation to the problem of controlling the oscillation of levels in the Great Lakes.

The principle laid down in Proposition II is at first sight inconsistent with equation (2). In discussing that equation it was seen that $f\,(h)$ increases or diminishes with h. But it has just been shown that a reduction of the fluctuations of h from its normal condition will increase the variation of Q, or $f\,(h)$. The explanation of this apparent paradox is, that any change in the normal variation of h^1 can only be brought about by some modification of the form or coefficients of $f\,(h)$; and that while h and $f\,(h)$ increase and decrease together for a fixed condition of the outlet, the rule has no application to a change from one condition of the outlet to another.

The converse of Proposition II is not necessarily true; that is, that an increase of oscillation in reservoir level, or of variation in outlet discharge, will diminish the variation or oscillation in the other. See remarks on artificial reservoirs under Proposition I.

III.—*In a series of reservoirs, such as has been described, if a permanent negative supply be introduced at any point, that is, if a portion of the supply be permanently withdrawn from any reservoir, it will cause a lowering of the mean level throughout the entire system below.*

This is apparent from equation (2) in which Q and h increase or diminish together. If, therefore, Q be permanently diminished, h must also be diminished, and the mean level of the reservoir will be lowered.

The lowering of the mean level in any reservoir, under the above assumption, will be such as to make $\frac{q}{c} = h'$, in which q is the negative supply and h' is the resulting fall in the mean level.

A permanent negative supply, however, unless of great relative magnitude, would have but little influence on the range of oscillation in the reservoir, provided the range of variation of S were not changed. If $f(h)$ were a right line, there would be no change in the oscillation, but if the function were of a form in which h appears at a higher power than unity, the oscillation of level would be increased by any lowering of the mean level.*

Under the assumption upon which Proposition III is based, the fall in mean level, resulting from the introduction of a negative supply, cannot be prevented by artificially increased storage in any part of the system. For assuming that $\int Q\,dt$ for the cycle is not to be changed, $\int + q\,dt$ must equal $\int - q'\,dt$, in which q and q' are the rates at which storage is accumulated during part of the cycle, and expended during the remainder. The total supply to the reservoir below is therefore not changed, and any increase during one period can be secured only by a corresponding decrease during a previous period. This might alter somewhat the oscillation of level, but would be powerless to prevent a lowering of level due to a permanent reduction of supply.†

Such a result can be prevented only by changing the form or coefficients of $f(h)$ in all outlets below the point where the negative supply is introduced—in other words, by contracting the outlets so as to diminish the normal discharge at mean level by an amount equal to the negative supply.

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* This may be a proper place to notice a fact which is of some interest as a theoretical refinement, although of no practical importance.

The mean level of a reservoir will rise as its fluctuations of level are diminished, and fall as the fluctuations are increased—the mean discharge remaining the same—and the mean level will stand highest when there is no fluctuation of level, or when the flow in the outlet is constant.

This law results from the nature of $f(h)$ in equation (2). If it were a right line, that is, if the variation in discharge per unit variation in h were the same at all points of h, then the mean level would always be the same for the same discharge. But $f(h)$ is nearly always a curve in which its value increases more rapidly than that of h. In the weir formulas, for example, h appears at the $\frac{3}{2}$ power, while in ordinary streams the parabolic function, ch^2, is true within narrow limits. It is obvious, therefore, that $\int f(h)\,dt$ for values of h above the mean level, must be greater than for the same values below, and since the mean discharge during the cycle cannot be increased, the plane of mean level must be somewhat below that corresponding to a constant discharge. In general, however, the difference of mean level would be inappreciable.

† Referring to the last preceding note, it will be seen that if the storage in the reservoir be so manipulated as to reduce the oscillation of levels in the reservoirs below, the mean level would be slightly raised; and if so manipulated as to increase the oscillations, the mean level would be slightly lowered. But such effects would ordinarily be infinitessimal in value.

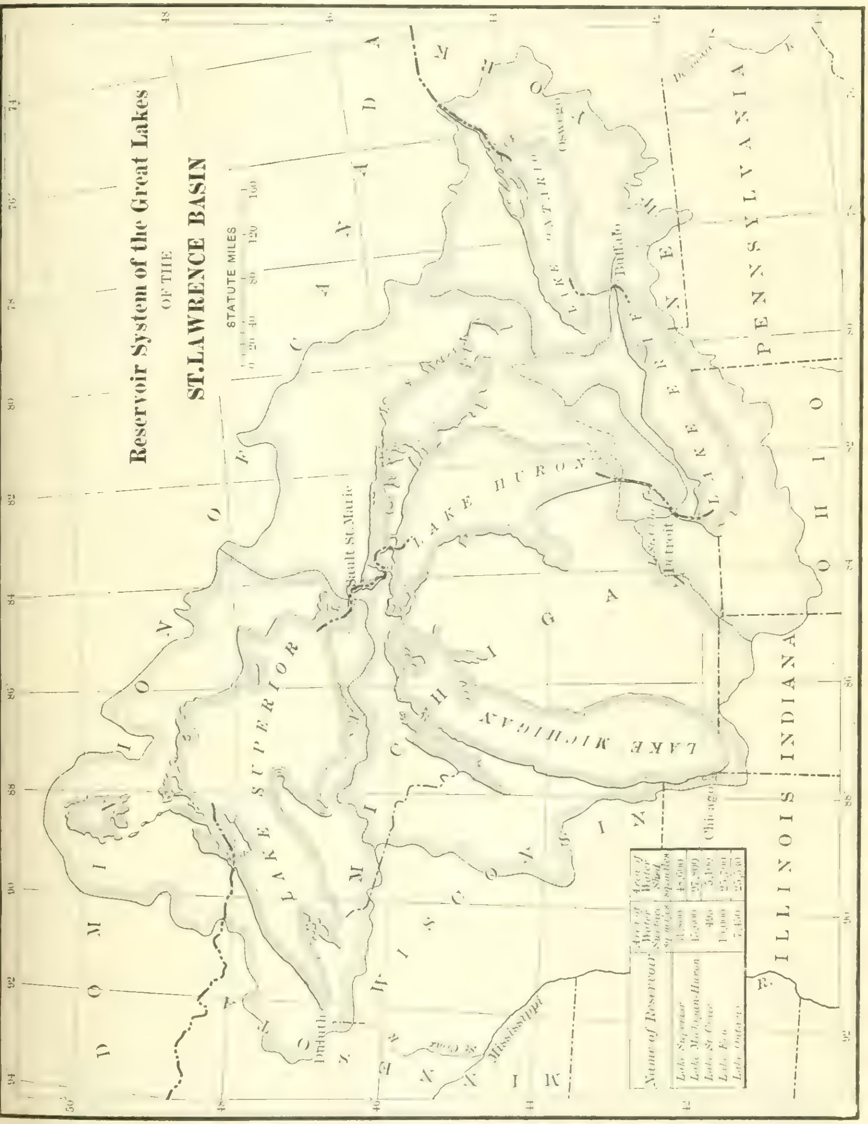

Reservoir System of the Great Lakes
OF THE
ST. LAWRENCE BASIN
STATUTE MILES
0 20 40 80 120 160
DOMINION OF CANADA
LAKE SUPERIOR
LAKE MICHIGAN
LAKE HURON
LAKE ONTARIO
LAKE ERIE
Sault St. Marie
Duluth
Chicago
Detroit
Buffalo
Oswego
MICHIGAN
WISCONSIN
MINN
NEW YORK
PENNSYLVANIA
OHIO
INDIANA
ILLINOIS
Mississippi
Name of Reservoir
Lake Superior
Lake Michigan-Huron
Lake St. Clair
Lake Erie
Lake Ontario

FIG. 2.

IV.—*Assuming the supply to remain unchanged, no modification of the normal oscillation of level in a reservoir, or of the normal variation of flow in the outlet, is possible except by some modification of the form or coefficients of* $f(h)$.

This proposition embodies the entire question of the character of works in the outlets of reservoirs designed to control or modify the normal oscillations of h and Q. Without entering at all into details of construction, it may be broadly stated that works of this class must take one of two forms, depending upon the immediate object to be attained. If that object be a reduction of oscillation of levels in the reservoir, then the controlling works must be of such a character as to increase the value of c [equations (3), (4), (5)]. If, on the other hand, the object be to restrict the variations of flow in the outlet, then the controlling works must be of such a character as to diminish the value of c. The first purpose may be accomplished by widening the outlet and diminishing the depth, as by the use of a long-crested weir around the head of the outlet, and the second by narrowing the outlet and increasing the depth.

Inasmuch as works of improvement, such as the deepening of natural channels, and works of simple contraction, such as wing dams, piers, etc., generally increase the depth and diminish the width, such works tend to uniformity in outlet discharge at the expense of uniformity of level in the reservoir.

Application of the Foregoing Propositions to the Reservoir System of the Great Lakes.

This system embraces five distinct reservoirs, the names and areas of which, together with the areas of the tributary water-sheds, are as follows:

Name.	Area of water surface.	Area of water-shed.
Lake Superior	31 800 square miles.	48 600 square miles.
Lake Michigan-Huron*	45 600 " "	97 800 " "
Lake St. Clair†	495 " "	5 100 " "
Lake Erie	10 000 " "	25 700 " "
Lake Ontario	7 450 " "	25 530 " "

* In this discussion Lakes Michigan and Huron will be considered as a single body of water.

† The areas in this table are taken from the report of the U. S. Deep Waterways Commission for 1896, except that the water-shed of Lake St. Clair, as given in that report, has been diminished by 1 220 square miles, and this area added to that of the Lake Erie water-shed. The water-shed map of the Commission report includes a considerable area in the St. Clair water-shed which would seem to drain into the Detroit River below a point where it could have any effect upon the levels of Lake St Clair.

The connecting channels are of considerable length and irregularity, but the retardation of effects, due to the time required for the water to flow through them, is probably in no case greater than twenty-four hours. It may be much less, if those portions are omitted which are practically inlets of the lakes, taking their stage directly from that of

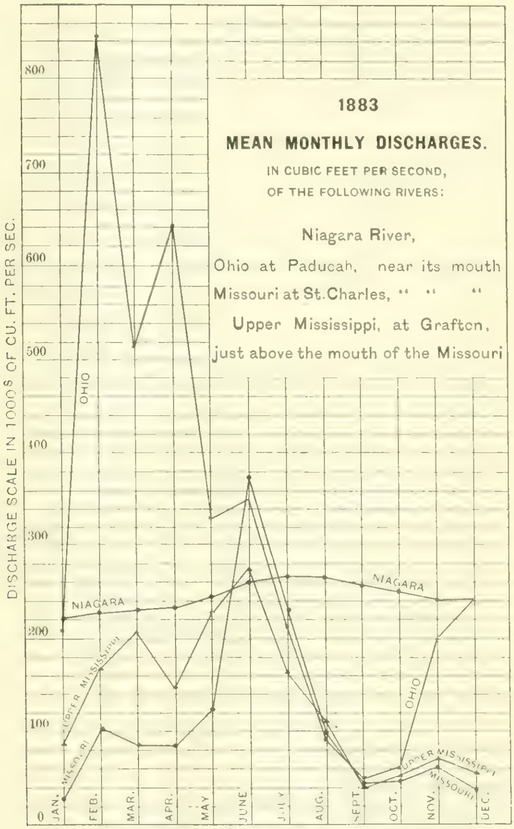


FIG. 3.

the lakes, and only those stretches are considered where the stage is dependent solely on the discharge.

Although existing data do not permit the precise determination of the values of c for the various outlets, they may be stated within a probable limit of error that will answer all the purposes of a rigid demonstration.

The values of *c*, here assumed, are as follows:

For Sault Ste. Marie River...........17 000 second-feet.
" St. Clair and Detroit Rivers......26 000 " "
" Niagara River....................30 000 " "
" St. Lawrence River..............34 000 " "

Deductions from Proposition I.

To appreciate fully the moderating effect of these reservoirs upon the stream flowing through them, as explained under Proposition I (page 530), it will be of interest to make a comparison with other well-known streams of similar magnitude. Take, for this purpose, the Ohio River at its mouth, the Missouri at its mouth, and the Mississippi at the mouth of the Missouri, and let them be compared with the Niagara at Buffalo. The areas of water-sheds of these four streams, and their mean discharge in second-feet, are as follows:*

	Niagara.	Ohio.	Missouri.	Mississippi.
Water-shed in square miles..............	265 095	205 750	530 810	171 570
Discharge in second-feet..................	232 800	307 000	100 000	130 000

The accompanying diagram, Fig. 3, exhibits graphically the variations in mean monthly discharge of these streams for the year 1883. The actual maxima and minima show, of course, a considerably larger divergence than the monthly means, the ratios $\frac{\text{(maximum discharge)}}{\text{(minimum discharge)}}$ for the above year being, for the Ohio, 28.22; for the Missouri, 29.00, and for the Mississippi, 10.29. For Niagara the ratio, disregarding wind effects, probably does not exceed, in any one year, 1.50.

In terms of absolute quantities, the storage by which this result is produced may be readily determined. The mean annual oscillation of level in Lake Superior, based upon 25 years' observations, is 0.93 ft.; in Lake Michigan-Huron, 1 ft., and in Lake Erie, 1.16 ft. These oscillations represent an aggregate storage of 2 419 billion cubic feet of water—greater than the over-flow excess of the Mississippi River at Cairo in the great flood of 1897, and equivalent to about 153 000 second-feet for a period of six months. The maximum annual oscillation is about twice the above mean, and, of course, represents twice the amount of storage.

* The above data for discharge are based upon 25 years' record (1871–1895) for Niagara, and 6 years' record (1880–1885) for the other streams.

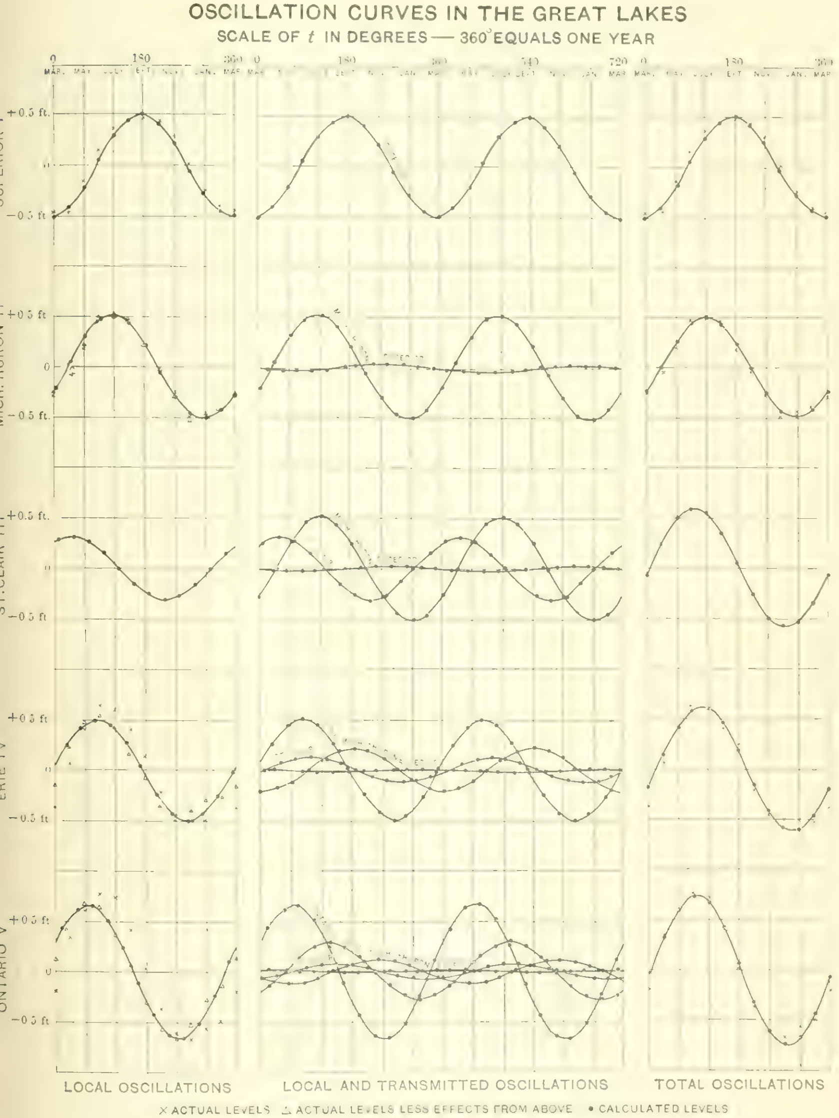

OSCILLATION CURVES IN THE GREAT LAKES
SCALE OF t IN DEGREES — 360° EQUALS ONE YEAR
+0.5 ft.
0
−0.5 ft.
LOCAL OSCILLATIONS
LOCAL AND TRANSMITTED OSCILLATIONS
TOTAL OSCILLATIONS
× ACTUAL LEVELS △ ACTUAL LEVELS LESS EFFECTS FROM ABOVE • CALCULATED LEVELS

FIG. 4.

Not less in magnitude and importance is the cyclic oscillation referred to earlier in this paper. During the period from 1872 to 1876, for example, the mean annual levels of the lakes above Niagara rose continuously, the aggregate storage being about 4 000 billion cubic feet. During the next three years the levels fell continuously, the aggregate loss of storage amounting to about 3 600 billion cubic feet.

The enormous moderating influence of these vast storage reservoirs thus practically eliminates, not only the regularly recurring changes in supply, but all sudden, erratic, or transient irregularities, and produces a flow in the outlets which, as already stated, is practically uniform from one year's end to the other.

In applying the formulas (3), (4) and (5), to the Great Lakes, no attempt has been made to go farther than to adapt them to the mean curves based upon 25 years' observations. To these, the ordinary sine curve, used in the analysis, has been fitted as closely as possible. The correspondence of the curves is not exact, but is near enough not to affect the general conclusions drawn from the discussion.

The following table gives the value of the retardation R, and the coefficients of reduction P and p, for the five lakes:

Name of lake.	R.	P.	p.
Superior	85.7	0.096	.....
Michigan-Huron	85.3	0.102	0.067
St. Clair	6.1	0.994	0.994
Erie	62.5	0.475	0.412
Ontario	51.3	0.635	0.560

This table, and the accompanying oscillation curves, Fig. 4, form a complete exhibit of the local and transmitted elements of the aggregate oscillation in each unit of the system.*

Column (2) explains the well known fact that the highest and lowest stages in the lakes occur long after the periods of maximum and minimum of supply have passed. However, from the curves and from column (2) the approximate mean dates of maximum or minimum supply for each of the lakes can be determined. To do this, take from the curve of oscillation due to local supply in any lake the dates of its maximum and minimum, and from these subtract the corresponding retardations in column (2). The results deduced by this process are

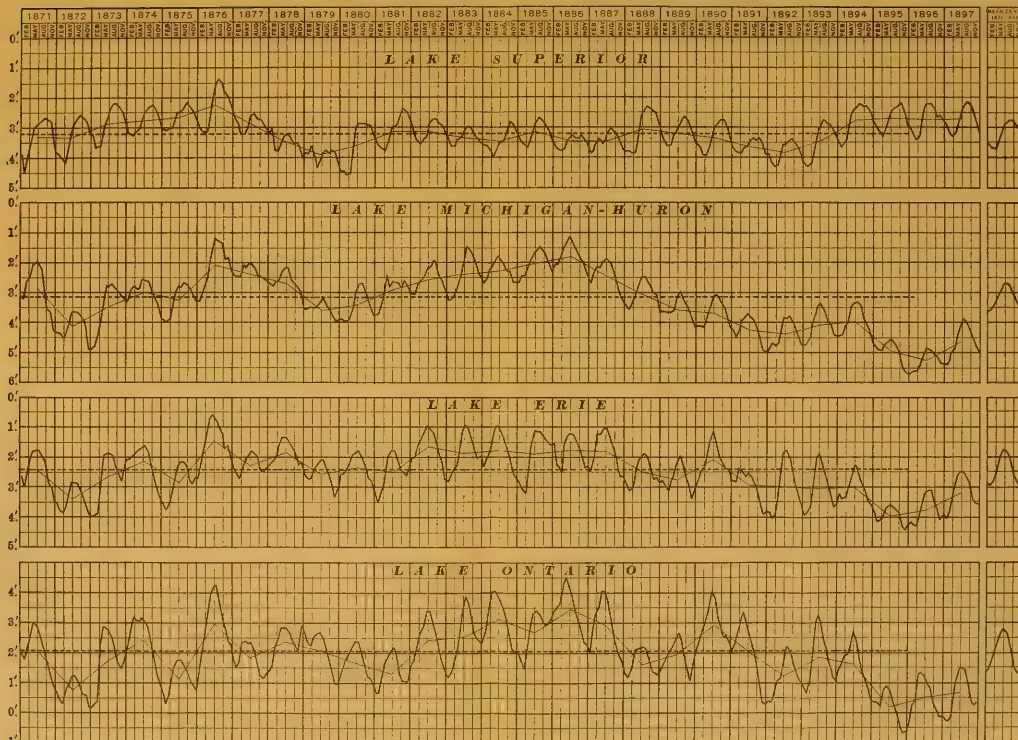
* The effect of the variable supply to Superior consumes 291 days in reaching Ontario, and its maximum is then only 1% of the total maximum oscillation in the latter lake.

PLATE XLIV.
PAPERS AM. SOC. C. E.
JUNE, 1898.
CHITTENDEN AND SEDDON
ON LEVELS OF THE GREAT LAKES.

CHART SHOWING THE OSCILLATIONS OF LEVEL AND OF MEAN ANNUAL LEVEL OF THE

GREAT LAKES OF THE ST. LAWRENCE BASIN

FROM 1871 TO 1897 INCLUSIVE, WITH MEANS OF TWENTY-FIVE YEARS FROM 1871 TO 1895 INCLUSIVE.



given in the table below, and fall well within the probable limits of actual occurrence.

Name of Lake.	Date of actual h max.	Date of local h max.	Date of actual h min.	Date of local h min.	Retardation.	Date of S max.	Date of S min.
Superior	Sept.	Sept.	Mar.	Mar.	86	June	Dec.
Michigan-Huron	July	July	Dec.	Jan.	85	April	Oct.
St. Clair	July	April	Dec.	Oct.	6	April	Oct.
Erie	June	June	Feb.	Dec.	63	April	Oct.
Ontario	June	May	Dec.	Nov.	51	April	Sept.

The quantity S, it must be borne in mind, is a very complex one. It is the result of at least three important and ever active causes—precipitation upon, and evaporation from, the surface of the reservoir, and run-off from the tributary water-shed. The first and third elements are always positive and the second always negative; but their maxima and minima are rarely coincident, and probably none of them coincide very closely with the actual aggregate. Precipitation, as based upon many years' observations, has two yearly maxima, occurring usually in June and September, and two minima, occurring usually in August and February. Run-off has its maximum generally in those months when the melting of the winter snows occurs. This is ordinarily in March for the lower lake region, and in April for the Lake Superior region. The minimum of run-off spreads over a pretty long period, ranging from September to February, and is controlled, not only by actual precipitation, but by the influence of cold weather in congealing precipitation and checking the flow of streams. The maximum of evaporation occurs generally in July and August, and the minimum in the period from December to February; but there are certain phenomena, to be referred to later on, which indicate a higher winter rate of evaporation than is generally supposed.

The complex and variable elements, of which the actual local supply for any of the lakes is made up, render it extremely hazardous to predict from them the time of actual maximum and minimum; for these may incline toward any of the component elements which happen at the time to predominate. No method seems so rational as that above applied of working backward from the known dates of maximum and minimum oscillations due to local supply.

Assuming the mean discharge of the outlets to be, for Superior, 75 000 second-feet; for Michigan-Huron, 195 000 second-feet; for St.

Clair, 200 000 second-feet; for Erie, 225 000 second-feet; for Ontario, 260 000 second-feet; the corresponding local supply for each of the lakes in thousands of second-feet per month, based upon the twenty-five year curves, is found to be as follows:

Month.	Superior.	Michigan-Huron.	St. Clair.	Erie.	Ontario.
January	− 2	+111	+19	+43	+62
February	− 31	+185	+21	+52	+74
March	+ 75	+235	+20	+55	+75
April	+115	+255	+15	+49	+66
May	+152	+240	+ 8	+37	+49
June	+164	+191	− 1	+23	+27
July	+153	+129	− 5	+ 7	+ 8
August	+119	+ 55	− 7	− 2	− 4
September	+ 75	+ 5	− 6	− 5	− 5
October	− 31	− 15	− 1	− 1	− 4
November	− 2	+ 0	+ 6	+13	+21
December	− 14	− 47	+13	+27	+43

The results here given are to be taken only as general approximations, in the absence of precise data as to the mean discharge of the outlets; but they serve to show the fact, intended to be brought out here, that at certain seasons there is a negative supply on all of the lakes. The cause will naturally be attributed to the preponderance of evaporation at such times over rainfall and run-off; but the perplexing feature of the phenomenon is that in one lake in particular—Superior—this negative supply comes in months when evaporation is supposed to be at a minimum. An examination of Table No. 2 will show that on all the lakes there are large deficiencies at times, even in winter months. No explanation suggests itself except that evaporation from ice and snow on the lakes may be greater than is commonly supposed.

Relative Effects of Local and Transmitted Supplies.—Plate III, showing oscillation curves, affords a striking illustration of the observation made in discussing Proposition I (page 530), viz., that, in the general case, the oscillation of levels will be controlled by the transmitted supply when the reservoir is relatively small, and by the local supply when it is relatively large. Thus, the maximum of oscillation in Michigan-Huron, if due to local supply alone, would be 4% greater than it actually is, and would occur about three days earlier. The maximum of oscillation, if due to the transmitted supply from Superior alone, would be only 7% as great as it actually is, and would occur 141 days later.

For St. Clair, the local maximum would be about 53% as great as the actual, and would occur about 57 days earlier; while the trans-

mitted maximum would be 83% as great as the actual, and would occur 32 days later.

For Erie, the local maximum would be about 80% as great as the actual and would occur about 20 days earlier; while the transmitted maximum would be about 39% as great as the actual and would occur 51 days later.

For Ontario, the local maximum would be about 88% as great as the actual, and would occur about 28 days earlier; while the transmitted maximum would be about 47% as great as the actual, and would occur about 64 days later.

From these figures, and from the curves, an excellent idea may be had of the parts which local and transmitted supplies play in the actual oscillations of the lakes. Except in St. Clair, it is seen that the local supply is the controlling factor, and that in one lake—Michigan-Huron—the transmitted supply actually diminishes the total fluctuation. In St. Clair the transmitted supply controls—as was to have been expected on account of its relatively small area,

It is also plain that any attempt to control the fluctuations of one lake by modifying the inflow from another will produce only insignificant results.

In like manner it is manifest that the oft-asserted mutual dependence of levels between Michigan-Huron and Erie, evidence of which, it is urged by some, is the fact that their difference of level scarcely ever varies as much as 1 ft.,* has no foundation in fact. St. Clair completely interrupts the continuity of flow on account of the fluctuation due to its own local supply.†

* Lake Michigan-Huron and Lake Erie rise and fall together, not because of any mutual dependence of levels, but because similar climatic forces are operating simultaneously in both basins. The fact that their levels do not move in closer unison—that sometimes Erie is falling when Michigan-Huron is rising, and *vice versa*—still further negatives the probability of any very intimate relation between the oscillations of level in these two bodies of water.

† The influence of St. Clair upon the discharge of the Detroit River may even be greater than has been here assumed. Unfortunately there are no actual data as to the oscillation of levels in this lake, and it has been necessary to deduce a curve from known data, such as the area of the lake, the inflow from Michigan-Huron, and an assumed S, based upon the similarity of climatic conditions on the watersheds of Michigan-Huron, Erie and St. Clair. In this, however, an error may have been made in assuming an eccentricity for the St. Clair curve of supply but little greater than that for the other lakes. The water-shed of St. Clair, compared with the lake area, is relatively much larger than those of the other lakes, as may be seen from the following ratios of lake to land area:

Superior	1.5
Michigan-Huron	2.1
St. Clair	10.3
Erie	2.4
Ontario	3.4

Remembering that S is the algebraic sum of precipitation on the lake surface, evaporation from it, and run-off from the water-shed, a little consideration of the character of these elements will show that as run-off predominates over rainfall and evaporation, the eccentricity of S will increase. It is quite possible, therefore, that the range of local supply for St. Clair has not been assumed large enough, and that the oscillations of level in that reservoir are due, to a greater extent than has been admitted, to local supply.

Deductions from Proposition II.

Table No. 2, appended to this paper, shows in detail, month by month for twenty-five years, and for each of the five lakes, the amount of storage accumulated or lost, both in terms of *h* and in thousands of second-feet per month. It also shows similar data for the mean of the twenty-five years. Table No. 3 shows the oscillation of mean annual level in feet in each of the lakes from 1871 to 1895, inclusive, with the equivalent storage in thousands of cubic feet for the period of one year. Table No. 4 shows the rate in second-feet at which storage corresponding to a rise or fall of 6 ins. on each of the lakes would have to be accumulated or expended in periods varying from one to three months. The tables show, not only the data for each lake by itself, but the cumulative effects in descending the series, supposing the causes to be operating simultaneously throughout the system.

These tables supply ample data for conclusions as to the practicability of controlling the levels of the lakes within fixed limits—say, 6 ins. Thus, let it be assumed that the outlet of any of the lakes, Superior, for instance, is to be so regulated that the annual oscillation of the lake level shall not exceed 0.5 ft. The amount of storage represented by a depth of 0.5 ft. on Lake Superior is equivalent to 168 000 second-feet for one month. Now, during the months from March to September, in 1871, for example, storage was accumulated in this lake at the following rates in thousands of second-feet: March, 146; April, 168; May, 178; June, 49; July, 24; August, 20; September, 34. Assuming that this storage is to be reduced to an equivalent of 168 000 second-feet for one month, and that the reduction is to be taken from the months of most rapid accumulation, it would give the following results: It would reduce the storage in March by 38, in April by 60, and in May by 70, and would leave for each of these months 108 000 second-feet which would have to be run out in addition to the normal discharge.* This, it will be understood, is based upon the impossible supposition that climatic conditions can be foreseen so as to apply the allowable storage of 0.5 ft. to those months of most rapid accumulation. In actual practice, of course, no such result could be realized, and the increase of outflow would be greater than above indicated.

Applying this process to the ensuing period of falling levels, from October, 1871, to April, 1872, it will be found that for the months of

December and January the outflow would have had to be reduced by 94 000 second-feet*, and for the other months, during which the lake was falling, by the full amounts given in the table. It requires no comment to show, not only the impracticability, but the utter impossibility of any such accomplishment.

However, the difficulty here encountered would evidently be less in the lower units of the system, if considered by themselves, for there the reservoir area is much less and the flow of the outlets much greater. To see what effect would be produced upon the flow of Niagara by a reduction to 0.5 ft. of the oscillation in Erie, apply the above process to that lake for the year 1871-72. It will be found that the outflow would not have been increased by more than 24 000 second-feet, nor restricted by more than 32 000 second-feet. There are other years, however, in which the increase of outflow would have been much greater, but it may be assumed that any practicable restriction of the oscillation of Erie to a limit of 0.5 ft. would probably not often increase or decrease the flow in the outlet by more than 30 000 second-feet.* Whether a change of this amount in the normal variation of flow of Niagara, particularly in the low-water period, could be admitted, is at least an open question, and one that would have to be settled before regulation could actually take place.

If regulation to a limit of 6 ins. were simultaneously applied to all the lakes, or even to Michigan-Huron and Erie, the cumulative effect on Niagara would render the project wholly impossible.

It will probably impress any one who carefully examines this subject that an interference with the normal annual oscillation of levels of the lakes will at best be an uncertain and hazardous undertaking; and that, if such interference were to take place, it might better be for the purpose of increasing, rather than diminishing, the oscillations of levels in the reservoir, and of thus reducing the variations of flow in the outlet.

It is, after all, not so much the annual oscillation that gives rise to complaint on the part of navigators as the long periodic oscillation. The real source of trouble is the continuous subsidence of levels through several years, by which the carrying power of the great ships is constantly reduced, and their earning capacity diminished. Can these periodic oscillations be eliminated, and can the mean annual levels be always maintained above a minimum plane?

* For values of mean discharge of the various lakes, see p. 21.

So far as the rise is concerned, the problem is certainly feasible, for the increased flow, due to its elimination, when spread over such long periods, would not be objectionable, even in its cumulative effects.

The case is not equally clear in regard to the subsidence of levels. An extreme example of the fall of the mean annual levels of Michigan-Huron and Erie is that of the year 1894–95. Michigan-Huron dropped during that period at a rate of 42 000 second-feet, and Erie at the rate of 8 220 second-feet; but Superior in the meantime rose by an average of 1 700 second-feet. To have prevented the subsidence of levels in Michigan-Huron and Erie would have required a uniform reduction of the normal discharge of Michigan-Huron by 40 300 second-feet, and of Erie alone by 8 220 second-feet, while the cumulative effect in Erie would have been 48 520 second-feet. This being a period when the normal discharge of these rivers was injuriously small, these reductions would probably not have been admissible during the navigation season. If restricted to one-third of a year, that is, to the winter season, they would have been 120 900 second-feet for Michigan-Huron, 24 663 second-feet for Erie alone, or 145 560 second-feet as the cumulative effect from Michigan-Huron and Erie. As the normal discharge of Michigan-Huron at this time was probably between 150 000 and 175 000 second-feet, and of Erie not much more than 200 000 second-feet, the elimination of the cyclic fluctuation alone would, it is seen, have involved a reduction of discharge in the outlets, during the entire winter season of four months, of nearly 75 per cent.

Should full investigation show that this amount of reduction in the discharge of the outlets during the non-navigation season could be permitted without injury to any interests, the possibility of uniform regulation, such as is here assumed, would still be dependent upon a condition which can never be realized. It is impossible to forecast climatic conditions for anything like the periods covered by these cycles. Those charged with the control of the outlets must always be for the most part in the dark as to what is going to happen in regard to rainfall and run-off. It is therefore clearly impossible to fix in advance, even for the period of a single year, a uniform rate at which the discharge of the outlets must be increased or diminished in order to prevent a change in the mean annual level. The only way to accomplish this result that suggests itself would be to accumulate an amount of storage in the lakes during each winter, and the ensuing

period of high water, equal to the maximum subsidence which experience shows may be expected to follow during the low-water season. If this were done, the levels could always be maintained above a minimum plane. This result, however, could be attained only at the cost of increasing now and then the annual oscillation.

Of course, to make any regulation of the lake levels practicable the mean levels must be raised somewhat, in order to be able to secure the increase of outflow which would at times be necessary; but this elevation of mean level is in itself one of the ends contemplated by the general scheme of lake level regulation. Taken by itself it is unquestionably a practicable matter, at least within small limits. Such a step would indeed involve grave difficulties in the outlets themselves, to which reference will presently be made, and will always be complicated with the question of damages, but so far as the simple matter of raising the levels of the lakes is concerned, there can be not the slightest doubt of its feasibility. It is in fact claimed that this has already been accomplished in Lake Superior by works placed in the outlet of St. Mary's River for other purposes. These works consist of bridge piers and water-power works, and the contraction of the outlet has been such as to give an estimated rise of 0.5 ft. in the level of the lake, prior to January, 1896.*

The question of permanently raising the levels of the lakes is, or may be made, entirely independent of that lake level regulation. A simple contraction of the outlets will accomplish the first purpose, but not the second. It would in fact increase the oscillation somewhat, in accordance with a principle brought out in discussing Proposition IV (page 540), that if the outlet of a reservoir be narrowed or deepened, the value of c will be diminished, and the oscillation of level will be increased while the variation in flow will be lessened.

Deductions from Proposition III.

The third of these general propositions relates to the effects upon the lake levels, of the diversion of any portion of the supply of the

* Report by E. E. Haskell, U. S. Asst. Engineer, dated January 11th, 1896. Although it is true that the level of Lake Superior was rising for several years prior to 1896, while the levels of the lower lakes were falling, still the climatic conditions are dissimilar enough to lead one to hesitate in attributing this particular rise to artificial causes alone.

Great Lakes to other watersheds, and to the methods of counteracting such effect. The specific application of the proposition now in view, is to the Chicago Drainage Canal with its contemplated diversion of 10 000 second-feet, and to the scheme advocated by some engineers of diverting a much larger amount, say 30 000 second-feet, to help out the low-water navigation of the Mississippi. That such diversions will permanently lower the levels of all the lakes, below and including that from which they are taken, is certain. The amount of this lowering is a function of c of the various outlets, and its correct determination depends upon the prior correct determination of this coefficient.

Among the suggestions advanced for counteracting this lowering effect is that of storing water in Lake Superior to make up the amount of diversion. A reference to the discussion of Proposition III (page 537) will show the fallacy of this scheme. The storage of water in Lake Superior must commence by cutting off the outflow. When the stored water is run out, the total increase of flow over the normal condition will only be equal to the previous decrease. In other words, the total supply to the lakes below, upon which their mean level depends, cannot be altered a particle by storage.

The only way the effects of diversions can be counteracted is to contract the outlets, so that the flow through them at the normal mean level shall be diminished by an amount equal to the diversion.

Analogous to the effect which must result from the permanent withdrawal of any portion of the supply of the lake system to other water-sheds is that due to an increase of the cross-sections of the outlets resulting from improvements in the interests of navigation. Such works must necessarily result in a permanent lowering of mean level in the reservoir above. The remedy is, of course, to contract the outlets sufficiently to diminish the outflow at the normal mean level by an amount equal to the increase caused by the enlargement of section. In superficial area the contraction would have to be greater than the previous enlargement, for the whole change thus made in the cross-section is in the direction of deepening and narrowing the outlet; and a deep and narrow channel will carry more water than a wide and shallow one of the same area of cross-section.

Deductions from Proposition IV.

All works intended to change the levels of the lakes, to control their oscillations, to counteract diversions, or to regulate the flow of the outlets, must be placed directly in, or at the head of, the outlets themselves. The two broad classes into which such works must fall have already been pointed out, viz.:

(*1*) Works tending to increase the width and diminish the depth, with the result of diminishing oscillation of level and increasing variations of discharge; and

(*2*) Works tending to contract the width and increase the depth, with the result of increasing oscillation of levels and diminishing variations in discharge.

The only example of a work under the first class is that proposed by some engineers for controlling the level of Lake Erie. For obvious reasons, the regulation of the levels of this lake is of more importance than that of any of the others. There need only be mentioned its vast harbor interests, and the fact that at least two, and probably three, canals will in the future lead directly from its lower extremity to Ontario or to the Hudson River. A fixed depth in these harbors and over the miter sills is of great importance. To secure uniformity of level it has been proposed to enclose the head of Niagara by a fixed weir, which will give the desired permanent increase in elevation of level, and which shall be of such length, that a change of depth on the crest of, say, 6 ins. will so modify the outflow as to prevent a rise or fall of level of more than that amount.

Two objections appear to be conclusive against this arrangement: (*a*) The very advantage which first commends it, that of automatic regulation of oscillations, would, in reality, prove a serious disadvantage; and (*b*) the whole result can be accomplished in a simpler, more effective, and less expensive way by works placed directly in the outlet.

The trouble with the automatic action of the long-crested weir is that it would always give the same discharge for the same depth of overflow; whereas, the best regulation of the discharge might require that it be not always the same for the same stage. It has been seen that a regulation of the levels of Erie within a limit of 6 ins. would increase

or decrease the normal flow in Niagara at times by as much as 30 000 second-feet; but the range of the normal variation itself above or below the mean doubtless frequently amounts to 30 000 second-feet. The weir must therefore be of such length that a variation of 6 ins. in the depth of flow over its crest will give a variation of discharge of 60 000 second-feet. When it is considered that changes of 2 to 4 ft. in gauge readings due to wind effects "are of so frequent occurrence as to be considered common,"* and that changes of from 4 to 8 ft. are not rare, the effect of a long weir upon the discharge of Niagara, in times of heavy winds, would, it is readily seen, be disastrous. With a rising lake the matter would be of less consequence; for if the level of the lake were regulated but a foot or two above its present mean stage, the increased overflow would choke up the outlet and drown out the weir; but with a falling lake, no such counteracting effect would be experienced, and Niagara River would frequently run dry.

As before stated, the whole result expected from the long weir can be obtained in a simpler, more effective and less expensive way, by works placed directly in the outlet. The form of structure which appears to be essential to a rational regulation of the levels of the lakes is one that shall be in a measure under the control of human agencies. Disclaiming any purpose of proposing technical or specific details of construction, the following general plan may be suggested as embodying what would seem to be the most practicable method of control. At a suitable section of the outlet, preferably where rock foundation may be had, let a series of piers, similar to bridge piers, be erected at proper intervals, whose aggregate area of cross-section below the water surface will be equal to the contraction of the channel section, which is necessary to secure the desired permanent elevation of the mean level of the lake. These piers would, thus, by themselves, accomplish that part of the purpose of regulation which relates to a permanent elevation of level. For the purpose of controlling the oscillations, some arrangement by which the area of the new cross-section can be enlarged or diminished is necessary. To accomplish this, let a sill be anchored to the bottom of the river from pier to pier, and let a bridge be laid from pier to pier, but slightly elevated above the water surface. Upon this bridge, at convenient intervals, let

* Report of the United States Deep Waterways Commission of 1896, p. 156.

proper arrangements be made for the handling of good-sized needles which may be lowered into the river and supported by the bridge at the upper end, and by the sill at the lower end. Or, better still, these needles could be fixed permanently in position, but balanced on the longitudinal axis, so that the broad flat surface or the narrow edge, as desired, could be presented to the current. These needles, when in place, need not be adjacent, but there may be open intervals between them; the main point being that their aggregate area shall be sufficient, by proper manipulation, to give the desired control over the flow in the outlet. A navigable pass of sufficient width should be left entirely free from obstruction. With an arrangement of this sort, all the advantages of the fixed weir may be realized, with the very important additional advantage of adaptation of control to actual conditions as they arise. Such a structure ought to be much cheaper than a long weir. Its care and management would cost something, but the expense from this cause ought not to be great.

One important drawback will always be encountered with any form of contraction that can be devised. The raising of the level of the lakes will increase the slope at the site of the works of contraction, and may develop currents of such rapidity as to interfere seriously with the passage of boats. At the Sault Ste. Marie this is a matter of less importance, for locks there are a necessity any way, and the only effect would be to increase the lift. Even this might not result if Michigan-Huron and Superior were each raised by a like amount. But in passing from Huron, St. Clair or Erie into the rivers below, the case is different, and contracting works might, and probably would, develop strong currents and sharp slopes in their immediate vicinity.* It is doubtful if the simultaneous and equal raising of Erie and Michigan-Huron would obviate the difficulty, for the increased channel depth would diminish the slopes, and would tend to concentrate the fall at the contracting works. It might be necessary to have a series of works, one below another, with the area of contraction gradually diminishing so as to let the slope down gently by distributing it over

* The outlets of both Michigan-Huron and Erie have engorged sections where they leave the lakes; and even in their natural conditions show marked slopes and high velocity of flow at these points.

a considerable distance. In any event, the use of locks would be too serious an inconvenience to merit consideration.*

Physical Data.

It will have been observed that, in this discussion, the primary data upon which the oscillation of lake levels depends—those, namely, of precipitation, evaporation, and run-off—have been dealt with scarcely at all. In fact S does not appear in equations (3), (4) and (5), but only the quantities A, h and c. The discussion of the reservoir question therefore depends upon a knowledge of the area of the lakes, the oscillation of the levels, and the characteristics of flow in the outlets. The first of these elements is known with precision. The second has been a matter of record for nearly forty years, and in this discussion is based upon a period of observation of twenty-five years. The data are not indeed all that could be wished. The records have not been made up from self-registering gauges, as would have been the better plan, but from observations taken at a particular hour each day. Considering the sudden effects of winds and barometric changes on the levels of the lakes, it is clear that the individual records cannot be of much value; but since erroneous values are liable to occur as much on one side of the truth as on the other, the monthly means of daily readings are probably not far from correct, while the mean of so long a period as twenty-five years must be very accurate.

Of the discharge of the outlets comparatively little is yet known. In this matter, however, it is not so much the absolute value of the mean discharge that is needed, as the variations from the mean. In other words, Q is of less importance than c. In the assumption of c it has been necessary to proceed somewhat in the dark, but nevertheless with confidence, because a considerable variation in c will not materially affect the results with reservoirs of such magnitude as the Great Lakes. In like manner the assumption that $f(h)$ is a right line is so near the truth, that its departure therefrom cuts no figure whatever in the general result. It is confidently believed, therefore, that the general conclusions here arrived at will not be essentially modified by more exhaustive data.

* It is scarcely necessary to invite attention to the international feature of this problem. No radical step in the direction of lake-level regulation can be taken, except with the co-operation, or at least consent, of Canada. It is, in fact, by no means improbable that this feature of the question may prove the most embarrassing of all.

It is, nevertheless, of great importance to a final solution of the problems of the lakes that these data be made as complete as possible. The future records of the lake levels should be kept with self-registering gauges, and there should be one of these for Lake St. Clair as well as for the other lakes. The discharge measurements should embrace every phase of flow in the outlets, and particular care should be exercised to secure simultaneous measurements at various stations along the channels connecting Huron with Erie. It is a matter of congratulation to know that these important steps are now in contemplation by the existing Deep Waterways Commission.

Concerning S, the collection of data appears to be altogether of secondary importance, and this for the excellent reason, that, having A, h, c and Q, S can be determined from them more accurately than in any other way. If the difficulties in the way of a satisfactory determination of any one of the elements of S—precipitation, evaporation, or run-off—are considered, it will be conceded that an approach to the problem from that direction is wholly injudicious. Take Lake Superior, for example. The precipitation at any one of the points, Duluth, Marquette, Sault Ste. Marie, White River or Port Arthur, around the shore on the lake, varies materially from that at any other point. Admitting that correct records can be kept at these points, how is the true proportion of lake area to be assigned to each in order to determine the average precipitation over the whole?

The question of evaporation is still more unsatisfactory. There is no practicable method of determining it accurately. How are the effects of winds on such great bodies of water to be determined? It is known that waves increase the exposed area about $2\frac{1}{2}\%$, while the spray from breakers adds still more; but at the same time the commotion of the water brings up the cooler depths to the surface and lowers the temperature upon which the evaporation largely depends.*

Again, observations taken near shore might not hold for the interior of the lake, where the atmosphere may reasonably be assumed to be more laden with moisture and to be of a lower temperature. The element of evaporation from ice and snow would be a very uncertain one, owing to the absence of any definite knowledge of the extent of ice-covered areas. With these inherent uncertainties in the problem

* In midsummer the surface temperature on Lake Superior was found by Geo. Y. Wisner, M. Am. Soc. C. E., to fall 5 or 6° after heavy winds.

of determining the rate of evaporation on the lakes, great confidence can hardly attach to any attainable results.

The run-off from the land area of the basin could be determined with tolerable accuracy for any one year, by a continuous gauging of a sufficient number of streams, but the results would be of little value for another year in which conditions might be very different.

It is therefore not apparent why this particular line of data should receive special attention. The complex expression which gives their combined effect is already integrated in the rise and fall of the lake levels and in the flow of the outlets, and if these are known, S can be determined from them better than it can ever be measured.

A MATHEMATICAL ANALYSIS OF THE INFLUENCE OF RESERVOIRS UPON STREAM-FLOW.

By James A. Seddon,* C. E.

The fluctuation in the levels of a system of lakes only differs in its magnitude from the filling and emptying of a lock or a reservoir under varying conditions of inflow and outflow, though, of course, the true level over such large areas is not so easily observed. The action of the winds will certainly greatly change the levels at different points, the surface will answer to difference in barometric pressure, and small tidal effects may be found, and even after the primary disturbance is removed, the whole body of water may have been set to rocking backward and forward like a pendulum. It is thus only in the mean of a large number of points that the true level may be determined, or, as in actual practice, in the average of a time long enough to eliminate these shorter variations; but, this true level determined, all its variations are results alone of the inflow and the outflow.

The inflow will be called supply, and designated by S, and the outflow, discharge, and designated by Q, each taken in cubic feet per second, and with A representing the area of the lake in square feet, h its level on a scale of feet and T time in seconds. The fundamental equation of its fluctuation is

$$(S - Q)\, d\, T = A\, d\, h \qquad \text{(I)}$$

In this equation, S, the supply of the lake, is made up of the run-off from the drainage basin surrounding it, which has its annual rise and fall, like the high and low water periods of a river in a similar basin. But, unlike the river, the direct rainfall on its large area gives it at times a much more concentrated supply, and undoubtedly now and then its actual levels are subject to finite changes in comparatively insignificant intervals of time; but averaged up, as these are in the mean monthly levels, the effect of this direct supply approaches the annual cycle, and for a number of years must closely correspond to the variation of normal rainfall, and, finally, from these positive elements of supply there must be taken the negative element of evaporation, which has perhaps of all of them the most regular variation with the seasons.

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Thus, while the actual supply in a given year can never be exactly represented by any regular variation, the general type of supply may very fairly be assumed as a recurrent annual oscillation, and taken in its simplest form, as $S_1 + a \sin t$; where S_1 is a mean value of supply and t is a time arc on a circle, whose whole circumference represents one year, while as $\sin t$ passes from its extreme values of $+1$ to -1, the extreme oscillation of supply, of course, corresponds to the value of $2a$.

Unlike supply in its arbitrary character, Q, the discharge from the lake is, of course, determined at any time solely by the value of h, or the level of its outlet at that time. Just what relation exists between Q and h is a matter of observation. Were the outlet a plain weir of length l, with H measuring the height above its crest, the relation would be $Q = 3.3\, l\, H^{\frac{3}{2}}$, while the form that has more generally been applied to the flow of rivers is $Q = C\, H^2$; where H again gives the level above an approximate zero of discharge, and C is a constant to be determined from the discharge observations. Either of these more general expressions, however, makes the solution of Equation I unnecessarily difficult, and in the small oscillation of a lake surface it is sufficient to take Q as made up of a constant element Q_1, the value of discharge at an assumed level, and a variable element $c\,h$, where h gives the surface at any time above or below this arbitrary level, and c is the average change in discharge there per foot of rise or fall. Where large discharges are dealt with, some 10 or 15 ft. above their zero level, there is little difference in a foot or so between the arc of the discharge curve and the straight line $c\,h$, and it would be hardly necessary to consider this, even in the flow of a river; while in the case of the flow out of the lake, where the accidental oscillation at its outlet may exceed the whole range of its true level, such a difference would seem to be an even more needless refinement. In what follows, therefore, the discharge at any time will be taken as $Q_1 + c\,h$, or a linear variation with the value of h at that time.

Substituting these values for S and Q in Equation I gives

$$[(S_1 - Q_1) + (a \sin t - c\,h)]\; d\,T = A\, d\,h.$$

And for the present neglecting the arbitrary constant $(S_1 - Q_1)$, which may always be made zero at any time by taking a suitable level, from which h is to be measured for that time, Equation I becomes

$$(a \sin t - c\,h)\; d\,T = A\, d\,h.$$

In this equation T is time in seconds, while t is an arc on a circle in which 2π is to represent one year. The relation between them is, therefore, given by the following proportion:

$$T : t = 86\,400 \times 365 \text{ (seconds in one year)} : 2\pi$$

or

$$T = \frac{86\,400 \times 365}{6.2832}\, t;$$

and

$$d\,T = \frac{86\,400 \times 365}{6.2832}\, d\,t.$$

At the same time, it is also convenient to change the unit of A from square feet to square miles, so that the equation becomes

$$(a \sin t - c\,h)\,\frac{86\,400 \times 365}{6.2832}\,d\,t = (5\,280)^2\,A\,d\,h$$

or

$$(a \sin t - c\,h)\,0.180036\,d\,t = A\,d\,h \ldots\ldots\ldots\ldots\ldots\ldots \text{(II)}$$

or

$$d\,h + \frac{0.18\,c}{A}\,h\,d\,t = \frac{0.18}{A}\,a \sin t\,d\,t$$

This is a well-known form, and is solved by multiplying through by

$$e^{\frac{0.18\,c}{A}t}$$

or

$$e^{\frac{0.18\,c}{A}t}\,d\,h + h\,\frac{0.18\,c}{A}\,e^{\frac{0.18\,c}{A}t}\,d\,t = \frac{0.18}{A}\,a \sin t\,e^{\frac{0.18\,c}{A}t}\,d\,t$$

Where the left-hand side of the equation is the complete differential of

$$h \times e^{\frac{0.18\,c}{A}t},$$

and the right contains only the single variable t and is easily integrated by parts as follows:

$$\frac{0.18}{A}\,a \int \sin t\,e^{\frac{0.18\,c}{A}t}\,d\,t = \frac{0.18}{A}\,a \int \sin t\,\frac{d\left(e^{\frac{0.18\,c}{A}t}\right)}{\frac{0.18\,c}{A}}$$

$$= \frac{0.18}{A}\,a \left[\frac{\sin t\,e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} - \frac{1}{\frac{0.18\,c}{A}}\int \cos t\,e^{\frac{0.18\,c}{A}t}\,d\,t\right]$$

And again:

$$\int \cos t\,e^{\frac{0.18\,c}{A}t}\,d\,t = \int \cos t\,\frac{d\left(e^{\frac{0.18\,c}{A}t}\right)}{\frac{0.18\,c}{A}}$$

$$= \frac{\cos t\, e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} + \frac{1}{\frac{0.18\,c}{A}} \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t.$$

Hence

$$\frac{0.18}{A}\,a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t = \frac{0.18}{A}\,a \left[\frac{\sin t\, e^{\frac{0.18\,c}{A}t}}{\frac{0.18\,c}{A}} - \right.$$

$$\left. \frac{\cos t\, e^{\frac{0.18\,c}{A}t}}{\left(\frac{0.18\,c}{A}\right)^2} - \frac{1}{\left(\frac{0.18\,c}{A}\right)^2} \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t \right]$$

And solving for $\frac{0.18}{A}\,a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t$, gives

$$\frac{0.18}{A}\,a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t \left(1 + \frac{1}{\left(\frac{0.18\,c}{A}\right)^2}\right)$$

$$= \frac{0.18}{A}\,a\, \frac{1}{\left(\frac{0.18\,c}{A}\right)^2} \left[\frac{0.18\,c}{A} \sin t - \cos t\right] e^{\frac{0.18\,c}{A}t}$$

or, finally,

$$\frac{0.18}{A}\,a \int \sin t\, e^{\frac{0.18\,c}{A}t}\, d\,t = \frac{0.18}{A}\,a \left[\frac{\frac{0.18\,c}{A}\sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2}\right] e^{\frac{0.18\,c}{A}t}$$

And the primitive of differential Equation II becomes

$$h\, e^{\frac{0.18\,c}{A}t} = \frac{0.18}{A}\,a \left[\frac{\frac{0.18\,c}{A}\sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2}\right] e^{\frac{0.18\,c}{A}t} \pm C \left\{ \begin{array}{l}\text{The constant of in-}\\ \text{tegration.}\end{array}\right.$$

or dividing by $e^{\frac{0.18\,c}{A}t}$

$$h = \frac{0.18}{A}\,a \left[\frac{\frac{0.18\,c}{A}\sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2}\right] \pm \frac{C}{e^{\frac{0.18\,c}{A}t}}$$

In this form it is plain that if the value of h is to be recurrent, or the surface movement is restricted to an annual oscillation, the term

$\dfrac{C}{e^{\frac{0.18\,c}{A}\,t}}$ must disappear, and the constant of integration must, therefore, be zero. In this case it will be called h_1 where uniformly

$$h_1 = \frac{0.18}{A}\,a\left[\frac{\frac{0.18\,c}{A}\sin t - \cos t}{1+\left(\frac{0.18\,c}{A}\right)^2}\right]$$

Here, also, for the excluded term $(S_1 - Q_1)$ to be zero, the mean supply, S_1, must be constant from year to year and equal to Q_1, a mean discharge, thus fixing the zero from which h is measured at a mean level. As, however, it is well known that the mean supply is not constant from year to year, and that it also passes through a more or less well-defined cycle covering a number of years, this term $(S_1 - Q_1)$ may be again written $(S_{11} - Q_{11}) + (a' \sin t' - c\,h')$, and this multiplied by $(0.18\,d\,t)$ equals $A\,d\,h'$, or a second additional element to the surface variation. Here, for all practical purposes, this longer cycle may be assumed as recurrent, with S_{11}, the mean supply, for the number of years equal to Q_{11}, the mean discharge, and the total variation of surface in that case made up of the sum of h_1 and h' measured from the mean level corresponding to Q_{11}.

In this longer cycle, however, the arc t' no longer corresponds to one year, but to a number of years n.

or while $\qquad \dfrac{T}{t} = \dfrac{86\,400 \times 365}{2\pi}$; $\dfrac{T}{t'} = \dfrac{n \times 86\,400 \times 365}{2\,\pi}$

and $\qquad t = n\,t'$, and $d\,t = n\,d\,t'$.

Hence the differential equation is

$$(a' \sin t' - ch')\; n \times 0.18\,d\,t' = A\,d\,h',$$

and its primitive is

$$h' = \frac{n \times 0.18}{A}\,a'\left[\frac{\frac{n \times 0.18\,c}{A}\sin t' - \cos t'}{1+\left(\frac{n \times 0.18\,c}{A}\right)^2}\right] \pm \frac{C}{e^{\frac{n \times 0.18\,c}{A}\,t'}};$$

in which the term involving the constant of integration now disappears, by taking, as may in general be done, a recurrent variation of h'.

And as the additional oscillation has been taken through a number of years, so also an additional oscillation through a fraction of the year might be taken, and in a sum of these oscillations h may thus

be made to express all the variations of the surface for any such system of regular variations in the monthly and the annual supplies. It is, however, true that the supply in any actual case may be far from regular, and while $(Q_{11} + c\,h)$ always expresses the actual discharge (the difference between the curve and line as noted being insignificant) still $S_{11} + a \sin t + a' \sin t'$, etc., may not so express the actual supply, unless such altogether arbitrary values may be assigned to S_{11} from time to time as will make the expression practically fit any form of irregular variation. It is, therefore, desirable finally to solve Equation I in its most general form with $(S_{11} - Q_{11})$ simply an arbitrary constant, having, of course, alternating + and — values, which at any period of irregularity may be represented by $\triangle S$. In addition to h_1 and h' there is then a third element of variation h'', given by an additional term of the form

$$\frac{0.18}{A}\int \triangle S\, e^{\frac{0.18\,c}{A}t}\, dt$$

or

$$h'' = \frac{\triangle S}{c} \pm \frac{C}{e^{\frac{0.18\,c}{A}t}}$$

And here to determine the general form of the constant of integration C, it is necessary to consider how a lake would rise or fall with such a change in its supply as that of the arbitrary constant $\triangle S$.

Substituting then in Equation II there results

$$(\triangle S - ch'')\, 0.18\, d\,t = A\, d\,h''$$

or

$$d\,t = \frac{A}{0.18\,c}\,\frac{d\,h''}{\frac{\triangle S}{c} - h''}$$

or

$$t = -\frac{A}{0.18\,c}\log_e\left(\frac{\triangle S}{c} - h''\right) + C$$

And if t be taken as zero where h'' is zero and $t_{''}$ represents any point of time in such an arbitrary period of t corresponding to the arbitrary value $\triangle S$,

then

$$C = \frac{A}{0.18\,c}\log_e\left(\frac{\triangle S}{c}\right)$$

and

$$t_o = \frac{A}{0.18\,c} \log_e \left[\frac{\frac{\triangle S}{c}}{\frac{\triangle S}{c} - h''} \right]$$

or

$$e^{\frac{0.18\,c}{A} t_o} = \frac{\frac{\triangle S}{c}}{\frac{\triangle S}{c} - h''}$$

or

$$h'' = \frac{\triangle S}{c} - \frac{\frac{\triangle S}{c}}{e^{\frac{0.18\,c}{A} t_o}}$$

From which it is seen that a general form for the constant of integration C is $- \frac{\triangle S}{c}$ which, of course, is zero in all the former cases where $\triangle S$ was assumed zero.

The complete equation may be now written as follows:

$$h = h_1 + h' + h'' = \frac{0.18}{A} a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right] + \frac{n \times 0.18}{A} a'$$

$$\times \left[\frac{\frac{n \times 0.18\,c}{A} \sin t' - \cos t'}{1 + \left(\frac{n \times 0.18\,c}{A}\right)^2} \right] + \frac{\triangle S}{c} \left[1 - \frac{1}{e^{\frac{0.18c}{A} t_o}} \right] \text{(III)}$$

In which t is a continuous angle on a scale in which 360° equal one year, and t_o is the length of a natural arc on the same scale, but taken only through the arbitrary periods in which values may be assigned to $\triangle S$; while again t' is a continuous angle on a scale in which 360° $= n$ years. In this form, in any lake, the function

$\frac{0.18}{A} a \left[\frac{\frac{0.18\,c}{A} \sin t - \cos t}{1 + \left(\frac{0.18\,c}{A}\right)^2} \right]$ may be computed once for all through

a cycle, and its oscillation then gives at once the ratio of the oscillation of h_1 to a. Or, from the variation of surface in the mean monthly levels the corresponding variation in the supply, expressed by a, is gotten from this simpler proportion. In the same way for h', the var-

iation of supply from year to year expressed by a' may be brought into a like proportion with the variation in the mean annual levels. While finally for given periods of $t_{,,}$ corresponding proportions may be made up that express immediately the ΔS for any irregularity.

The relation of the variation of level in any lake to its variable supply may be here left for the more general problem which includes not only this, but its resultant effect on a lake below, and so on through a chain of lakes; and not to needlessly extend the problem, it is desirable in this case to use the simplest possible form for h. It is plain that in so far as h_1 and h' are concerned, it is only necessary to use the term h_1, for what will be generally true of the short cycle will be equally true of the long, and all equations deduced from the first can be immediately converted into equations for the second by substituting in the place of (0.18) the value of $(n\ 0.18)$ throughout, remembering, of course, that degrees in the angles t' are also to be taken as covering n times as many days as they do in the angles t; but the wholly different character of the term for h'' makes it necessary to exclude it altogether or to consider its effects down entirely as a question by itself.

This last has in fact been done, but it is all so much a matter of the particular case, that it will not be included in this study. Indeed, from the extreme where ΔS is assumed a permanent change in the mean supply, it is evident at once that in time any number of lakes would all come to about the same difference in level, represented by $\frac{\Delta S}{c}$; though it may be noted that none could absolutely reach this level until $t_{,,}$ became infinite. And while in very long periods there are practically no differences in the effects of this function on lakes below, at the same time in very short periods it has practically no effect whatever on them, so that in the range of ordinary irregularities in supply, it may simply be taken as rapidly disappearing in its effects on the lower lakes, but exactly how rapidly would depend on the length of the irregularity. It is only perhaps in the event of a system of storage in an upper lake through one season to be let out at another that the subject would have any general interest and there its effects are best worked out as a special case.

The more general problem of the effects of variable supplies on the levels of a chain of lakes is therefore confined to the consideration of the recurrent yearly oscillations h_1 and its effects below; and the

following general system of notation is taken for the resulting surface movements in a chain of n lakes.

$$
\begin{aligned}
&I h_1 \\
&I h_2 = II h_2 \\
&I h_3 = II h_3 = III h_3 \\
&I h_4 = II h_4 = III h_4 = IV h_4 \\
&I h_5 = II h_5 = III h_5 = IV h_5 = V h_5 \\
&\text{etc.,} \quad \text{etc.,} \quad \text{etc.,} \quad \text{etc.,} \quad \text{etc.,} \\
&I h_n = II h_n = III h_n = IV h_n = V h_n \text{ etc. } [n]\, h_n.
\end{aligned}
$$

Here the vertical lines represent the effects of the primary supply in the basin of each respective lake all the way down, and the horizontal represent all the effects that as a whole make up the surface movement of any lake in the system. In correspondence with this the areas of the respective lakes will be represented by A_1, A_2, etc., A_n, the "a" in the variations of supply $a \sin t$ by a_1, a_2, etc., a_n, and the change of discharge per foot of rise or fall in the outlets by C_1, C_2, etc., C_n.

Considering, now, the supply in the basin of the first lake and its effects down, there results:

$$I h_1 = a_1 \times \frac{0.18}{A_1} \times \frac{1}{1 + \left(\frac{0.18\, c_1}{A_1}\right)^2} \left[\frac{0.18\, c_1}{A_1} \sin t - \cos t\right]$$

and as the factors $\frac{0.18}{A}$ and $\frac{1}{1 + \left(\frac{0.18\, c}{A}\right)^2}$ will recur continually in these operations, it is convenient to represent them by the more simple terms K and V, writing them for their respective values of A and C, as K_1, K_2; V_1, V_2, etc.

With this substitution

$$I h_1 = a_1\, K_1\, V_1\, (K_1\, c_1 \sin t - \cos t)$$

and as for the second lake $c_1 \times h_1$ is the resulting variation of supply and $c_2 \times h_2$, the corresponding variation in its discharge, from equation II there results:

$$[a_1\, c_1\, K_1\, V_1\, (K_1\, c_1 \sin t - \cos t) - c_2\, h_2]\, 0.18\, d\, t = A_2\, d\, h_2$$

or

$$d\, h_2 + \frac{0.18\, c_2}{A_2} h_2\, d\, t = \frac{0.18}{A_2} a_1\, c_1\, K_1\, V_1\, (K_1\, c_1 \sin t - \cos t)\, d\, t$$

or

$$h_2\, e^{K_2 c_2 t} = a_1\, c_1\, K_1\, K_2\, V_1 \int (K_1\, c_1 \sin t - \cos t)\, e^{K_2 c_2 t}\, d\, t.$$

And in general, following the process of the former integration by parts,

$$\int e^{Kct} \sin t \, dt = \frac{Kc \sin t - \cos t}{1 + (K c)^2} e^{Kct}$$

and

$$\int e^{Kct} \cos t \, dt = \frac{Kc \cos t - \sin t}{1 + (K c)^2} e^{Kct}$$

so that

$$h_2 e^{K_2 c_2 t} = a_1 c_1 K_1 K_2 V_1 V_2 \left[K_1 c_1 (K_2 c_2 \sin t - \cos t) - (K_2 c_2 \cos t + \sin t)\right] e^{K_2 c_2 t}$$

or

$$I h_2 = a_1 c_1 K_1 K_2 V_1 V_2 \left[(K_2 c_2 K_1 c_1 - 1) \sin t - (K_2 c_2 + K_1 c_1) \cos t\right].$$

And again for the third lake with $c_2 \times h_2$ supply, and $c_3 \times h_3$ discharge, $h_3 e^{K_3 c_3 t} = a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 \int [(K_2 c_2 K_1 c_1 - 1) \sin t -$

$(K_2 c_2 + K_1 c_1) \cos t] e^{K_3 c_3 t} dt = a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 V_3 [(K_2 c_2 K_1 c_1 - 1)(K_3 c_3 \sin t - \cos t) - (K_2 c_2 + K_1 c_1)(K_3 c_3 \cos t + \sin t)] e^{K_3 c_3 t}$

or $I h_3 =$

$$a_1 c_1 c_2 K_1 K_2 K_3 V_1 V_2 V_3 \Big[\{K_3 c_3 (K_2 c_2 K_1 c_1 - 1) - (K_2 c_2 + K_1 c_1)\} \sin t - [K_3 c_3 (K_2 c_2 + K_1 c_1) + (K_2 c_2 K_1 c_1 - 1)] \cos t\Big]$$

and in general calling*

$$I h_n = a_1 \left|c\right|^{n-1} \left|K\right|^n \left|V\right|^n [I \alpha_n \sin t + I \beta_n \cos t] \ldots\ldots (IV)$$

in which α and β are formed as follows:

	$I \alpha =$	$I \beta =$
$I h_1 =$	$(K_1 c_1)$	$= -1$
$I h_2 =$	$(K_2 c_2 K_1 c_1 - 1)$	$= -(K_2 c_2 + K_1 c_1)$
$I h_3 =$	$K_3 c_3 (K_2 c_2 K_1 c_1 - 1) - (K_2 c_2 + K_1 c_1)$	$= -[K_3 c_3 (K_2 c_2 + K_1 c_1) + (K_2 c_2 K_1 c_1 - 1)]$

and finally

$$I h_n = K_n c_n \alpha_{n-1} + \beta_{n-1} \ldots\ldots\ldots\ldots\ldots\ldots = K_n c_n \beta_{n-1} - \alpha_{n-1}.$$

In the case of $II h_2$ begin with

$$II h_2 = a_2 K_2 V_2 (K_2 c_2 \sin t - \cos t)$$

and in the same way finally in the n^{th} lake there results

$$II h_n = a_2 \left|c_2\right|^{n-1} \left|K_2^n\right| \left|V_2^n\right| [II \alpha_n \sin t + II \beta_n \cos t]$$

and so on from lake to lake.

From these equations the effect of one lake upon another, and the total surface movements in any lake, are easily computed. Thus to

* In this paper the braces | | are used exclusively as a "factorial sign."

illustrate, the case of a chain of five lakes may be taken, each with a surface area of 10 000 square miles, with a 100 000 cu. ft. per second variation in the supply, entering each lake as a simultaneous oscillation, and with outlet capacities downward of, respectively, 10, 15, 20, 25 and 30 thousands of cubic feet per second per foot of rise or fall. Then

$$A_1 = A_2 = A_3 = A_4 = A_5 = 10\,000$$

$$a_1 = a_2 = a_3 = a_4 = a_5 = 50\,000$$

$$C_1 = 10\,000;\ C_2 = 15\,000;\ C_3 = 20\,000;\ C_4 = 25\,000;\ C_5 = 30\,000.$$

Hence

$$K C_1 = 0.18;\ K C_2 = 0.27;\ K C_3 = 0.36;\ K C_4 = 0.45;\ K C_5 = 0.54$$

and

$I\,\alpha_1$ .... $= +0.18$
0.27
126
036
$+0.0486$
-1.0000
$I\,\alpha_2$ .... $= -0.9514$
0.36
57084
28542
-0.3425
-0.4500
$I\,\alpha_3$ .... $= -0.7925$
0.45
39625
31700
-0.3566
$+0.7894$
$I\,\alpha_4$ .... $= +0.4328$
0.54
17312
21640
$+0.2337$
$+1.1477$
$I\,\alpha_5$ .... $= +1.3814$

$I\,\beta_1$ .... $= -1.0$
0.27
-0.27
-0.18
$I\,\beta_2$ .... $= -0.45$
0.36
270
135
-0.1620
$+0.9514$
$I\,\beta_3$ .... $= +0.7894$
0.45
39470
31576
$+0.35523$
$+0.7925$
$I\,\beta_4$ .... $= +1.1477$
0.54
45908
57385
$+0.6198$
-0.4328
$I\,\beta_5$ .... $= +0.1870$

and so for $II\,\alpha_2$, $II\,\beta_2$, to $II\,\alpha_5$, $II\,\beta_5$, etc., giving

α_1	α_2	α_3	α_4	α_5	
+ 0.1800	— 0.9514	— 0.7925	+ 0.4328	+ 1.3814	*I*
	+ 0.2700	— 0.9028	— 1.0363	+ 0.0597	*II*
		+ 0.3600	— 0.8380	— 1.2625	*III*
			+ 0.4500	— 0.7570	*IV*
				+ 0.5400	*V*

β_1	β_2	β_3	β_4	β_5	
— 1.0000	— 0.4500	+ 0.7894	+ 1.1477	+ 0.1870	*I*
	— 1.0000	— 0.6300	+ 0.6193	+ 1.3707	*II*
		— 1.0000	— 0.8100	+ 0.4006	*III*
			— 1.0000	— 0.9900	*IV*
				— 1.0000	*V*

Again the following are the logs of the general factor $a\left\{c\right\}^{n-1}\left\{K\ V\right\}^{n}$ which are easily made up by repeated additions

1	2	3	4	5	
$\{\ \}$	$\{\ \}$	$\{\ \}$	$\{\ \}$	$\{\ \}$	
$\bar{1}.940394$	$\bar{1}.165108$	$\bar{2}.543547$	$\bar{2}.019764$	$\bar{3}.561849$	*I*
	$\bar{1}.923683$	$\bar{1}.302122$	$\bar{2}.778940$	$\bar{2}.321024$	*II*
		$\bar{1}.901318$	$\bar{1}.377535$	$\bar{2}.919620$	*III*
			$\bar{1}.874157$	$\bar{1}.416242$	*IV*
				$\bar{1}.843114$	*V*

and multiplying here the respective values of α and β by the corresponding general factors and representing the products by $I\left\{\alpha_1\right\}$, $I\left\{\beta_1\right\}$, &c., there results finally

$\{\alpha_1\}$	$\{\alpha_2\}$	$\{\alpha_3\}$	$\{\alpha_4\}$	$\{\alpha_5\}$	
+ 0.15692	— 0.13915	— 0.02770	+ 0.00453	+ 0.00504	*I*
	+ 0.22649	— 0.18101	— 0.06229	+ 0.00125	*II*
		+ 0.28683	— 0.19988	— 0 10492	*III*
			+ 0.33680	— 0.19740	*IV*
				+ 0.37628	*V*

$\{\beta_1\}$	$\{\beta_2\}$	$\{\beta_3\}$	$\{\beta_4\}$	$\{\beta_5\}$	
− 0.87176	− 0.06581	+ 0.02760	+ 0.01201	+ 0.00068	*I*
	− 0.83885	− 0.12632	+ 0.03723	+ 0.02871	*II*
		− 0.79674	− 0.19321	+ 0.03329	*III*
			− 0 74844	− 0.25815	*IV*
				− 0.69681	*V*

Taking now t at intervals of 30° corresponding with the twelfth of the year and practically with the monthly means:

t	sin t	cos t
0°	0	+1
30°	+0.5	+0.866
60°	+0.866	+0.5
90°	+1	0
120°	+0.866	−0.5
150°	+0.5	−0.866
180°	0	−1

And with these values, from the equation $h = \{\alpha\} \sin t + \{\beta\} \cos t$, the whole cycle of each of these variations is readily made up as follows:

$t =$	0°	30°	60°	90°	120°	150°	180°
$I \{\alpha_1\} \sin t =$	+0.0000	+0.0785	+0.1359	+0.1569	+0.1359	+0.0785	+0.0000
$I \{\beta_1\} \cos t =$	−0.8718	−0.7550	−0.4359	−0.0000	+0.4359	+0.7550	+0.8718
$I h_1 =$	−0.8718	−0.6765	−0.3000	+0.1569	+0.5718	+0.8335	+0.8718
and with signs reversed $t =$	+ 210°	+ 240°	− 270°	300°	− 330°	− 360°	

completing the cycle.

These values for the half cycle are here given in series from the primary supply on, to its effect in the last lake.

	0°	30°	60°	90°	120°	150°	180°
$I h_1$	−0.8718	−0.6765	−0.3000	+0.1569	+0.5718	+0.8335	+0.8718
$I h_2$	−0.0658	−0.1266	−0.1534	−0.1391	−0.0876	−0.0126	+0.0658
$I h_3$	+0.0276	+0.0101	−0.0102	−0.0277	−0.0378	−0.0377	−0.0276
$I h_4$	+0.0120	+0.0127	+0.0099	+0.0045	−0.0021	−0.0081	−0.0120
$I h_5$	+0.0007	+0.0031	+0.0047	+0.0050	+0.0041	+0.0019	−0.0007
$II h_2$	−0.8388	−0.6133	−0.2233	+0.2265	+0.6155	+0.8397	+0.8388
$II h_3$	−0.1263	−0.1999	−0.2200	−0.1810	−0.0936	+0.0189	+0.1263
$II h_4$	+0.0372	−0.0011	−0.0353	−0.0622	−0.0725	−0.0633	−0.0372
$II h_5$	+0.0287	+0.0255	+0.0155	+0.0012	−0.0133	−0.0243	−0.0287
$III h_3$	−0.7967	−0.5466	−0.1500	+0.2868	+0.6468	+0.8334	+0.7967
$III h_4$	−0.1932	0.2672	−0.2697	−0.1999	−0.0765	+0.0674	+0.1932
$III h_5$	+0.0333	−0.0237	−0.0743	−0.1049	−0.1075	−0.0813	−0.0333
$IV h_4$	−0.7484	−0.4798	−0.0825	+0.3368	+0.6659	+0.8166	+0.7484
$IV h_5$	−0.2581	−0.3223	−0.3000	−0.1974	−0.0418	−0.1249	+0.2581
$V h_5$	−0.6968	−0.4154	−0.0225	+0.3763	+0.6743	+0.7916	+0.6968

And these values summed algebraically for their respective lakes would give in each case the total of the surface movement there, but it is easier to sum at once the coefficients of sin t and cos t and calculate directly the cycles of these resultant oscillations.

Giving the following values:

	0°	30°	60°	90°	120°	150°	180°
I h_1	—0.8718	—0.6765	—0.3000	+0.1569	+0.5718	+0.8335	+0.8718
I to II h_2	—0.9047	—0.7398	—0.3767	+0.0873	+0.5279	+0.8272	+0.9047
I to III h_3	—0.8955	—0.7364	—0.3801	+0.0781	+0.5153	+0.8146	+0.8955
I to IV h_4	—0.8924	—0.7332	—0.3777	+0.0792	+0.5147	+0.8124	+0.8924
I to V h_5	—0.8922	—0.7326	—0.3766	+0.0802	+0.5156	+0.8128	+0.8922

These values are shown graphically on Fig. 1, and while they show the general form of the oscillations, it is very evident that they do not give, with any degree of accuracy, the extremes. It is seen, of course, that while a sin t, or the supply, is in all cases at its maximum at 90°, its corresponding rise reaches its maximum in the first lake somewhere between the 150° and 180° periods, and that this again as a supply is once more retarded in the rise of the second lake, and so on from lake to lake; but the periods and the values of these maxima require additional calculations.

The value of t at these points of maxima or minima is, of course, determined by the condition $\frac{dh}{dt} = 0$, and in the general form of the equation

$$h_n = a \left\{c\right\}^{n-1} \left\{K\right\}^{n} \left\{V\right\}^{n} \left[\alpha_n \sin t + \beta_n \cos t\right]$$

$$\frac{d\,h_n}{d\,t} = 0, \text{ gives } \alpha_n \cos t = \beta_n \sin t, \text{ or } \tan t = \frac{\alpha}{\beta}$$

The two angles in the cycle differing by 180°, that correspond to this value of the tan t, are, of course, the points of maximum and minimum respectively; and by substituting either in the equation for h the + or — extreme is given. It is, however, enough simply to consider the maximum, as before only the half cycle was calculated, every value of h having, of course, a corresponding value with an opposite sign after a period of 180°.

As the effects of a primary supply are traced down, the retardation throws the maximum into later and later quadrants, and these must be followed by the successive change of signs in the term $\frac{\alpha}{\beta}$.

Thus, in the case of I, the original supply, a sin t, has its maximum as noted at 90°, while for I h_1, $\frac{\alpha_1}{\beta_1}$ is minus, and is therefore in the

second quadrant, and its angle 10° 12′ gives the absolute position as 180° 00′ — 10° 12′ = 169° 48′, and substituting this angle for t gives the value of 0.8858 for the maximum of $I\,h_1$. Again for $I\,h_2$, $\frac{\alpha_2}{\beta_2}$ is plus, and is thus in the third quadrant, and its angle 64° 41′ gives its position as 180° 00′ + 64° 41′ = 244° 41′, and its value 0.1539. In the same way $\frac{\alpha_3}{\beta_3}$ is minus and lies in the fourth quadrant, and its angle 45° 07′ places it at 360° 00′ — 45° 07′ = 314° 53′ with the maximum value 0.0391. While finally $\frac{\alpha_4}{\beta_4}$ and $\frac{\alpha_5}{\beta_5}$ are both plus and lie in the fifth quadrant at the points respectively 360° 00′ + 20° 40′ = 380° 40′ and 360° 00′ + 82° 18′ = 442° 18′, and maxima of 0.0128 and 0.0051.

The positions of these maxima and their values through this whole series of oscillations are tabulated as follows:

Angle t of maximum.

h_1	h_2	h_3	h_4	h_5	
169° 48′	244° 41′	314° 53′	380° 40′	442° 18′	*I*
	164° 53′	235° 05′	300° 52′	362° 30′	*II*
		160° 12′	225° 58′	287° 36′	*III*
			155° 46′	217° 24′	*IV*
				151° 38′	*V*

Value of maximum h.

h_1	h_2	h_3	h_4	h_5	
0.8858	0.1539	0.0391	0.0128	0.0051	*I*
	0.8689	0.2207	0.0726	0.0287	*II*
		0.8468	0.2780	0.1101	*III*
			0.8207	0.3250	*IV*
				0.7919	*V*

From these values of t it is seen at once that the difference between the maxima of $I\,h_1$ and $I\,h_2$, or 244° 41′ — 169° 48′ = 74° 53′ is also the difference between 164° 53′ and 90°, or between the maximum of supply, a sin t, and its rise, $II\,h_2$. And, again, 70° 12′ is the general difference between h_2 and h_3; 65° 46′ + between h_3 and h_4, and 61° 38′ between h_4 and h_5; which are also in each case the difference between 90°, the maxima of the respective supplies, a sin. t, and their corresponding oscillations in the first lake. It is then plain that the retardation in any lake is a constant dependent upon the physical proper-

ties of that lake, and in no way upon the magnitude of the oscillations passing through it. And just as it has been seen that any oscillation passing from a given lake to the next is retarded by a fixed time ; so it may be seen from the corresponding maximum values of h that they are also reduced by a fixed percentage.

It is time therefore to turn from the series of equations which have just been considered, to determine the formulas for this retardation and reduction from lake to lake ; for it is very evident that they are a much more general expression for the laws of lake movements than equations which simply give direct values for assumed times and supplies.

First, for the reduction ratio or percentage, which will be called p ; noting that it is enough to express it in the case of I from h_1 to h_n, for it is also the same from lake to lake in the case of II, III, etc. ; and omitting, therefore, the I in the general equation, and representing by h' and t' respectively the maximum rise and the time corresponding to it, there results:

$$h'_n = a \left\{c_1\right\}^{n-1} \left\{K_1\right\}^n \left\{V_1\right\}^n \left[\alpha_n \sin t' + \beta_n \cos t'\right] \dots\dots\dots\dots \quad (1)$$

and

$$\frac{\alpha_n}{\beta_n} = \frac{\sin t'}{\cos t'} \dots\dots\dots\dots\dots\dots\dots\dots \quad (2)$$

or from (2)

$$\sin t' = \frac{\alpha_n}{\sqrt{\alpha_n^2 + \beta_n^2}} \text{ and } \cos t' = \frac{\beta_n}{\sqrt{\alpha_n^2 + \beta_n^2}}$$

and eliminating t' between equations (1) and (2)

$$h'_n = a \left\{c_1\right\}^{n-1} \left\{K_1\right\}^n \left\{V_1\right\}^n \sqrt{\alpha_n^2 + \beta_n^2}.$$

Again from the law of formation in the general values of α and β, there results:

$\alpha_1 = K_1 c_1$ or $\alpha_1^2 = \overline{K_1 c_1}^2$. $\beta_1 = -1$; $\beta_1^2 = 1$. Hence, $\alpha_1^2 + \beta_1^2 = 1 + \overline{K_1 c_1}^2$

And again:

$$\alpha_2 = K_2 c_2 \alpha_1 + \beta_1 \text{ or } \alpha_2^2 = \overline{K_2 c_2}^2 \alpha_1 + 2 K_2 c_2 \alpha^2_1 \beta_1 + \beta_1^2$$

$$\beta_2 = K_2 c_2 \beta_1 - \alpha_1 \text{ or } \beta_2^2 = \overline{K_2 c_2}^2 \beta_1^2 - 2 K_2 c_2 \alpha_1 \beta_1 + \alpha_1^2$$

$$\text{Hence, } \alpha_2^2 + \beta_2^2 = \overline{K_2 c_2}^2 \left(\alpha_1^2 + \beta_1^2\right) + \alpha_1^2 + \beta_1^2$$

or

$$\alpha_2^2 + \beta_2^2 = \left(\alpha_1^2 + \beta_1^2\right)\left(1 + \overline{K_2 c_2}^2\right) = \left(1 + \overline{K_1 c_1}^2\right)\left(1 + \overline{K_2 c_2}^2\right)$$

and so on, until in general

$$\alpha_n^2 + \beta_n^2 = (1 + \overline{K_1 c_1}^2)\,(1 + \overline{K_2 c_2}^2)\,(1 + \overline{K_3 c_3}^2)\ \ldots.\ (1 + \overline{K_n c_n}^2) = \left\{1 + \overline{K_1 c_1}^2\right\}^n$$

from which finally there is obtained—

$$\frac{h'_n}{h'_{n-1}} = \frac{a\,\{c_1\}^{n-1}\,\{K_1\}^n\,\{V_1\}^n\,\sqrt{\left\{1 + \overline{K_1 c_1}^2\right\}^n}}{a\,\{c_1\}^{n-2}\,\{K_1\}^{n-1}\,\{V_1\}^{n-1}\sqrt{\left\{1 + \overline{K_1 c_1}^2\right\}^{n-1}}} = p_n$$

Or

$$p_n = c_{n-1}\,K_n\,V_n\sqrt{1 + \overline{K_n c_n}^2}$$

And writing now for K_n and V_n their respective values:

$$K_n = \frac{0.18}{A_n}$$

$$V_n = \frac{1}{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}$$

$$p_n = \frac{0.18\,c_{n-1}}{A_n}\,\frac{\sqrt{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}}{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2} = \frac{\frac{0.18\,c_{n-1}}{A_n}}{\sqrt{1 + \left(\frac{0.18 c_n}{A_n}\right)^2}} \ldots\ldots (V)$$

A value depending simply upon the area of the n^{th} lake and the discharging capacity of its inlet and its outlet.

Again, taking also the case I for the retardation, which will be called R; and representing by t'_{n-1} and t'_n, the time of the maxima in the $n-1$ and the n^{th} lake, respectively, there results:

$$\sin R_n = \sin\,(t'_n - t'_{n-1}) = \sin t'_n \cos t'_{n-1} - \cos t'_n \sin t'_{n-1}$$

and as before

$$\sin t_n^1 = \frac{\alpha_n}{\sqrt{\alpha_n^2 + \beta_n^2}} = \frac{\alpha_n}{\sqrt{\left\{1 + \overline{K_1 c_1}^2\right\}^n}} \text{ and } \cos t'_n = \frac{\beta_n}{\sqrt{\alpha_n^2 + \beta_n^2}} = \frac{\beta_n}{\sqrt{\left\{1 + \overline{K_1 c_1}^2\right\}^n}}$$

Hence,

$$\sin R_n = \frac{\alpha_n\,\beta_{n-1}}{\left\{1 + \overline{K_1 c_1}^2\right\}^{n-1}\sqrt{1 + \overline{K_n c_n}^2}} - \frac{\alpha_{n-1}\,\beta_n}{\left\{1 + \overline{K^1 c_1}^2\right\}^{n-1}\sqrt{1 + \overline{K_n c_n}^2}}$$

and as $\qquad \alpha_n\,\beta_{n-1} = (K_n\,c_n\,\alpha_{n-1} + \beta_{n-1})\,\beta_{n-1}$

and $$\alpha_{n-1}\,\beta_n = (K_n\,c_n\,\beta_{n-1} - \alpha_{n-1})\,\alpha_{n-1}$$

$$\alpha_n\,\beta_{n-1} - \alpha_{n-1}\,\beta_n = \alpha^2_{n-1} + \beta^2_{n-1} = \left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}$$

therefore,

$$\sin R_n = \frac{\left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}}{\left\{1 + \overline{K_1\,c_1}^2\right\}^{n-1}\sqrt{1 + \overline{K_n\,c_n}^2}} = \frac{1}{\sqrt{1 + \overline{K_n\,c_n}^2}} =$$

$$\frac{1}{\sqrt{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}} \ldots\ldots\ldots\ldots\ldots\ldots \quad (VI)$$

From equation (VI) it is seen that in passing through a given lake, oscillations of supply are retarded by this period, which depends alone upon the area and the outlet of that lake; but on comparing it with equation (V) it is evident at once that p_n is not a reduction of the same general character. It gives the ratio $\frac{h'_n}{h'_{n-1}}$, and is a most convenient function for calculating the effects of a given oscillation in any lake upon all the lakes that follow it; for at any point $h'_{n-1} \times p_n = h'_n$; but with exactly the same oscillation of supply in the n^{th} lake, and, in consequence, the same value of h'_n, p_n may have any number of values, as the oscillation from above which gives this supply has different outlets.

Thus finally there may be considered the reduction effected by a given lake on the variations of supply; or a reduction of the same general character as the retardation; and calling $\triangle\,S'_n$ and $\triangle\,Q'_n$ the maxima for the oscillations of supply and discharge respectively in the n^{th} lake, this absolute percentage of reduction which will be called P is given by the ratio $\frac{\triangle\,Q'_n}{\triangle\,S'_n}$.

This is easily determined from equation (V),

for $$h'_n c_n = \triangle\,Q'_n \text{ and } h'_{n-1}\,c_{n-1} = \triangle\,S'_n$$

or $$\frac{\triangle\,Q'_n}{\triangle\,S'_n} = \frac{h'_n}{h'_{n-1}} \times \frac{c_n}{c_{n-1}}$$

and as $$p_n = \frac{h'_n}{h'_{n-1}} = \frac{\frac{0.18\,c_{n-1}}{A_n}}{\sqrt{1 + \left(\frac{0.18\,c_n}{A_n}\right)^2}}$$

there results

$$p_n = \frac{c_n}{c_{n-1}} \times \frac{\dfrac{0.18\, c_{n-1}}{A_n}}{\sqrt{1 + \left(\dfrac{0.18\, c_n}{A_n}\right)^2}} = \frac{\dfrac{0.18\, c_n}{A_n}}{\sqrt{1 + \left(\dfrac{0.18\, c_n}{A_n}\right)^2}} \ldots\ldots\ldots (VII)$$

Thus in equations (*VI*) and (*VII*) may be seen finally, from the area and the outlet of the lake itself, its whole effect upon any oscillations of supply. It simply retards it by this given time, and reduces it to this given percentage.

Indeed, these last equations might have been originally determined simply from the consideration of the general relations between an oscillation of supply, $a \sin t$, and its corresponding oscillation of level.

Or as before

$$h = a \frac{\dfrac{0.18}{A}}{1 + \left(\dfrac{0.18\, c}{A}\right)^2} \left[\frac{0.18\, c}{A} \sin t - \cos t\right]$$

and again taking t' as the angle corresponding to the maximum h' there results from $\dfrac{dh}{dt} = 0$

$$-\frac{0.18\, c}{A} \cos t' = \sin t'; \text{ and } \sin t' = \frac{\dfrac{0.18\, c}{A}}{\sqrt{1 + \left(\dfrac{0.18\, c}{A}\right)^2}}$$

and

$$h \sin t' = a \frac{\dfrac{0.18}{A}}{1 + \left(\dfrac{0.18\, c}{A}\right)^2} \left[\frac{0.18\, c}{A} (\sin t \sin t' + \cos t \cos t')\right]$$

or

$$h \times \frac{\dfrac{0.18\, c}{A}}{\sqrt{1 + \left(\dfrac{0.18\, c}{A}\right)^2}} = a \times \frac{\dfrac{0.18}{A}}{1 + \left(\dfrac{0.18\, c}{A}\right)^2} \times \frac{0.18\, c}{A} \cos (t - t')$$

and finally

$$h = \frac{\dfrac{0.18}{A}}{\sqrt{1 + \left(\dfrac{0.18\, c}{A}\right)^2}} a \sin (90^\circ + t - t') \ldots\ldots\ldots\ldots\ldots\ldots (VIII)$$

Which, simply reduced by the factor $\dfrac{\dfrac{0.18}{A}}{\sqrt{1 + \left(\dfrac{0.18\, c}{A}\right)^2}}$, is alto-

gether a similar oscillation to $a \sin t$, only shifted to a new origin of time, with a zero of 90° before the maximum t', or zero where $t' - 90° = t$, as the origin in the original, $a \sin t$, was zero 90° before its maximum: and as this is true of any oscillation in any lake, it is just as applicable to the n^{th} oscillation as the first: and the formulas for reduction and retardation determined from it apply to all.

From equation (*VIII*) it is very evident that the maximum of level is given in all cases by the equation

$$h' = \frac{\dfrac{0.18}{A}}{\sqrt{1 + \left(\dfrac{0.18\,c}{A}\right)^2}}\,a;$$

where a represents the maximum of any supply oscillation from any source.

And writing f for this function $\dfrac{\dfrac{0.18}{A}}{\sqrt{1 + \left(\dfrac{0.18\,c}{A}\right)^2}}$,

and for a given lake in a series f_n, there results $\dfrac{h'_n}{f_n} = a_n$, by which, from the observed maximum of rise h'_n in any case, the corresponding supply may be at once determined; while, again, this is easily carried down through any number of lakes by their successive values of the retardation and the reduction ratio.

An analysis of the mean oscillations of the Great Lakes is here given to illustrate the application of the above formulas to the study of such data.

In this case the following data are given :

	A.	*C.*	*Log f.*	*Log p.*	*t′.*	*R.*
I. Superior	31 800	17 000	$\bar{6}.7505$	......	174° 30′	84° 30′
II. Michigan-Huron	45 600	26 000	$\bar{6}.5944$	$\bar{2}.8245$	174° 08′	84° 08′
III. St. Clair	495	26 000	$\bar{5}.5826$	$\bar{1}.9976$	96° 02′	6° 02′
IV. Erie	10 000	30 000	$\bar{5}.1995$	$\bar{1}.6147$	151° 38′	61° 38′
V. Ontario	7 450	34 000	$\bar{5}.2711$	$\bar{1}.7482$	140° 36′	50° 36′

The means of twenty-five years, from 1871 to 1895, for Superior are as follows:

Observed oscillations.			Computed trial oscillations.			
				t	Ih_1	t
September.	+0.47	+0.47	September...	174° − 30°	+0.500	174° + 30°
August.....	+0.45	+0.44	October.....	144° − 30°	+0.433	204° + 30°
July........	+0.37	+0.29	November...	114° − 30°	+0.250	234° + 30°
June.......	+0.16	+0.01	December...	84° − 30°	0.000	264° + 30°
May........	−0.15	−0.25	January.....	54° − 30°	−0.250	294° + 30°
April.......	−0.45	−0.40	February....	24° − 30°	−0.433	324° + 30°
March......	−0.46	−0.46	March........	− 06°	−0.500	354°

By plotting and comparing this trial oscillation with the observed one, it is seen that the zero of its t corresponds closely with the mean March period, and that its range fits the whole series of observed values about as well as possible.

The value of the maximum of Ih_1 is therefore taken as 0.50, and the oscillation of its supply is therefore—

$$
\begin{aligned}
& \qquad \log \\
0.50 &= \bar{1}.6990 \\
f_1 &= \underline{\bar{6}.7505} \\
a_{,} &= 4.9495 \quad = 83\,480. \qquad \text{or } I\,S = 83\,480 \sin t
\end{aligned}
$$

The effects of this upon all the lakes below is readily carried down as follows:

Maximum Effects.

					log.
			Ih'_1		$= \bar{1}.6990 = 0.500$
It'_1		= 174° 30′		$+ \log p_2$	$\underline{\bar{2}.8245}$
	$+ R_2$	$\underline{84°\ 08'}$			
It'_2		= 258° 38′	Ih'_2		$= \bar{2}.5235 = 0.033$
	$+ R_3$	$\underline{6°\ 02'}$		$+ \log p_3$	$\underline{\bar{1}.9976}$
It'_3		= 264° 40′	Ih'_3		$= \bar{2}.5211 = 0.033$
	$+ R_4$	$\underline{61°\ 38'}$		$+ \log p_4$	$\underline{\bar{1}.6147}$
It'_4		= 326° 18′	Ih'_4		$= \bar{2}.1358 = 0.014$
	$+ R_5$	$\underline{50°\ 36'}$		$+ \log p_5$	$\underline{\bar{1}.7482}$
It'_5		= 376° 54′	Ih'_5		$= \bar{3}.8840 = 0.008$

Again, from these times and values of the maxima, the complete oscillations are as follows:

Michigan-Huron.		St. Clair.		Erie.		Ontario.	
t.	Ih_2.	t.	Ih_3.	t.	Ih_4.	t.	Ih_5.
259° − 30°	+0.033	265° − 30°	+0.033	326° − 30°	+0.014	377° − 30°	+0.008
229° − 30°	+0.029	235° − 30°	+0.029	296° − 30°	+0.012	347° − 30°	+0.007
199° − 30°	+0.017	205° − 30°	+0.017	266° − 30°	+0.007	317° − 30°	+0.004
169° − 30°	+0.000	175° − 30°	0.000	236° − 30°	0.000	287° − 30°	0.000
139° 30	−0.017	145° − 30°	−0.017	206° − 30°	−0.007	257° − 30°	−0.004
109° − 30°	−0.029	115° − 30°	−0.029	176° − 30°	−0.012	227° − 30°	−0.007
79°	−0.033	85°	−0.033	146°	−0.014	197°	−0.008

Again, from the monthly means of the Michigan-Huron data, the effect there of the oscillation from Superior is first subtracted, and the result gives the oscillation of Michigan-Huron from the supply of its own water-shed. This is called the local observed oscillation, and with its corresponding computed trial oscillation is given as follows:

Local observed oscillation.				Computed trial oscillation.		
				t.	Ih_2	t.
July.........	+0.52	+0.52	July........	174°	+0.520	174°
June.........	+0.49	+0.46	August,.....	144°	+0.450	204°
May..........	+0.23	+0.22	September.	114°	+0.260	234°
April.........	−0·03	−0.03	October,....	84°	0.000	264°
March,........	−0.27	−0.29	November..	54°	−0.260	294°
February....	−0.42	−0.53	December..	24°	−0.450	324°
January.....	−0.48	−0.48	January...	− 06°	−0.520	354°

Plotting and comparing these oscillations, it is seen that the maxmum of 0.52 is best fitted to the whole series, from which—

$$\begin{aligned} & \qquad \log \\ 0.52 &= \bar{1}.7160 \\ f_2 &= \bar{6}.5944 \\ a_2 &= 5.1216 = 129\ 470 \end{aligned} \qquad \text{or } HS = 129\ 470 \sin t.$$

The zero of t, however, must in this case be put some 4° later than the mid-January period; and to bring here the computed values of this oscillation and its effects below to the same time scale as Superior, 56°

must in all cases be subtracted from its values of t. This, however, may be done at once on the retardations, and then the Michigan-Huron effects on the Superior time scale are as follows:

MAXIMUM EFFECTS.

Michigan-Huron time.	$II\ t_2$	$174°\ 08'$			log
		$-\ 56°$			
Superior time.	$II\ t'_2$	$=118°\ 08'$		$II\ h'_2$	$\bar{1}.7160 = 0.520$
	$+R_3$	$6°\ 02'$		$+\log p_3$	$\bar{1}.9976$
	$II\ t'_3$	$=124°\ 10'$		$II\ h'_3$	$=\bar{1}.7136 = 0.517$
	$+R_4$	$61°\ 38'$		$+\log p_4$	$\bar{1}.6147$
	$II\ t'_4$	$=185°\ 48'$		$II\ h'_4$	$=\bar{1}.3283 = 0.213$
	$+R_5$	$50°\ 36'$		$+\log p_5$	$\bar{1}.7482$
	$II\ t'_5$	$=236°\ 24'$		$II\ h'_5$	$=\bar{1}.0765 = 0.119$

And as before:

COMPLETE OSCILLATIONS.

ST. CLAIR.		ERIE.		ONTARIO.	
t.	$II\ h_3$.	t.	$II\ h_4$.	t.	$II\ h_5$.
124°	+0.517	186°	+0.213	236°	+0.119
94°	+0.448	156°	+0.184	206°	+0.103
64°	+0.259	126°	+0.107	176°	+0.060
34°	0.000	96°	0.000	146°	0.000
04°	—0.259	66°	—0.107	116°	—0.060
— 26°	—0.448	36°	—0.184	86°	—0.103
— 56°	—0.517	06°	—0.213	56°	—0.119

In the case of St. Clair there is lacking the observed oscillation to start with, but this may be closely approached by assuming, for its local supply a value of a_3 corresponding to its water-shed, and summing the effects of this with the effects from above.

In this case a_3 was finally taken as most probably about 8 000 with the same period as the Michigan-Huron supply, giving $S = 8\ 000 \sin t$, where the t is reckoned on the Michigan-Huron scale.

Here then

$$III\ h'_3 = a_3 f_3 \left\{ \begin{array}{lll} & & \log \\ 8\ 000 & = & 3.9031 \\ f_3 & = & \bar{5}.5826 \\ III\ h'_3 & = & \bar{1}.4857 = 0.306; \end{array} \right.$$

and the following are the

Maximum Effects.

Michigan-Huron time. $III\ t'_3$	$= 96°\ 02'$		log	
	$56°$			
Superior time. $III\ t'_3$	$= 40°\ 02'$	$III\ h'_3$	$= \bar{1}.4857$	$= 0.306$
$+R_4$	$61°\ 38'$	$= \log\ p_4$	$\bar{1}.6147$	
$III\ t'_4$	$= 101°\ 40'$	$III\ h'_4$	$= \bar{1}.1004$	$= 0.126$
$+R_5$	$50°\ 36'$	$+ \log\ p_5$	$\bar{1}.7482$	
$III\ t'_5$	$= 152°\ 16'$	$III\ h'_5$	$= \bar{2}.8486$	$= 0.070$

And as before

Complete Oscillations.

St. Clair.		Erie.		Ontario.	
t.	$III\ h_3$.	t.	$III\ h_4$.	t.	$III\ h_5$.
40°	+0.306	102°	+0.126	152°	+0.070
70°	+0.265	132°	+0.109	122°	+0.061
100°	+0.153	162°	+0.063	92°	+0.035
130°	0.000	192°	0.000	62°	0.000
160°	—0.153	222°	—0.063	32°	—0.035
190°	—0.265	252°	—0.109	02°	—0.061
220°	—0.306	282°	—0.126	—28°	—0.070

In this case in the place of the computed trial oscillation, the complete oscillation of St. Clair is taken to add to the effects from above.

For Erie, again subtracting the effects of the three lakes above from its mean levels, the following is left for the local observed oscillation and the computed oscillation corresponding to it.

Local observed oscillation.				Computed Trial Oscillation.		
				t	$IV\ h_4$	t
June	+0.545	+0.545	June.......	152	+0.500	152
May.......	+0.470	+0.415	July	122	+0.433	182
April......	+0.215	+0.165	August....	92	+0.250	212
March	—0.145	—0.075	September	62	0.000	242
February .	—0.275	—0.355	October ...	32	—0.250	272
January...	—0.295	—0.490	November.	02	—0.433	302
December.	—0.395	—0.395	December .	—28	—0.500	332

And accepting this trial oscillation, there results

$$\left.\begin{array}{rcl} & & \log \\ 0.50 & = & \bar{1}.6990 \\ f_4 & = & \bar{5}.1995 \\ a_4 & = & 4.4995 = 31\,580 \end{array}\right\} \text{ or } IV\,S = 31\,580 \sin t.$$

While plotting and comparing it is seen that the zero of t is some six degrees earlier than the mid-January period, or in this case 66° must be subtracted from all values of t, to reduce them to the Superior time scale.

Hence:

Maximum effects.					Complete oscillation. Ontario.	
Erie time.					t	$IV\,h_5$
	$IV\,t'_4$	= 151° 38′			136	+0.280
		66° 00′		log	106	+0.243
Superior time.	$IV\,t'_4$	= 85° 38′	$IV\,h'_4$	= $\bar{1}.6990$ = 0.500	76	+0.140
	$+R_5$	50° 36′	+log p_5	$\bar{1}.7482$	46	0.000
	$IV\,t'_5$	= 136° 14′	$IV\,h'_5$	= $\bar{1}.4472$ = 0.280	16	—0.140
					— 14	—0.243
					— 44	—0.280

Again, for Ontario, we have

Local Observed Oscillation.				Computed Trial Oscillation.		
				t	$V\,h_5$	t
May........	+0.665	+0.665	May........	136	+0.665	136
April.......	+0.555	+0.640	June	106	+0.576	166
March	+0.130	+0.365	July........	76	+0.333	196
February ..	—0.135	+0.075	August....	46	0.000	226
January...	—0.275	—0.300	September	16	—0.333	256
December.	—0.530	—0.575	October....	—14	—0.576	286
November.	—0.665	—0.665	November.	—44	—0.665	316

From which

$$\left.\begin{array}{rcl} & & \log \\ 0.665 & = & \bar{1}.8228 \\ f_5 & = & \bar{5}.2711 \\ a_5 & = & 4.5517 = 35\,620 \end{array}\right\} \text{ or } V\,S = 35\,620 \sin t.$$

While plotting and comparing it is seen that the zero of its t falls 14° earlier than the mid-January period, or subtracting 74° from the t of its trial oscillation brings it also upon the Superior time scale.

In going down the chain of lakes, out of these different elements of their oscillations any desired combination can be readily made up by scaling coincident values of h and taking their algebraic sums. Of these the leading combinations have been made up and shown with the observed variation of the mean levels on Fig. 2 ; and, indeed, it is mainly in their graphic representation that this series of movements is best followed. However, the purpose of this study has been simply the analysis of these movements, and with the equations and the illustrations that have here been given, it is thought that there are few questions that may arise in regard to variations of supply and their effects through any chain of lakes or reservoirs that may not be readily calculated.

In contrast with these relations in the lake levels it may be interesting to note briefly the character of similar relations in the Mississippi River. There, of course, the mathematical basis is wanting. Between the lakes, it matters not what energy may be stored in the flow, it is wholly absorbed through agitation in the inert mass below. Thus the sheer fall of the Niagara River makes not the slightest difference in the relation of supply to discharge, or the rise of one lake to the other ; but it would certainly be a bold assumption to start with, to hold that essentially the same thing was true for the river ; and even if that were the case, it is known that the linear variation of Q to h is not correct through the great range of its flood oscillations.

While, therefore, we cannot here calculate between two points what should be the retardation R, the reduction P, or the ratio of rise p, there are at some twenty or more locations along the Lower Mississippi twenty-five to forty years of daily gauge-readings from which to determine what they are ; and as all these data have been worked over, again and again, along just these lines, its positive showings at least merit a comparison with the above theoretic deductions.

They may be stated as follows: First, for the ratio of reduction P, as might be expected, it is practically unity. If there is any general decrease in the magnitude of the discharge as the flood-wave passes down, it is altogether beyond the range of observation. However, p, the ratio of rise or fall has in general a marked difference in value between different gauges. It is a well-determined constant for each reach of the river ; and while in the course of a number of years it may change to some extent with a change of regimen, in the lower Missis-

sippi it is remarkably stable. Finally, R, the retardation, is also there a constant of the reach. Its value is very different from gauge to gauge, and differs greatly per mile in different parts of the river, but once determined for a given reach it seems to be practically the same there for all floods and for all time. It is, however, measured in days and hours, where the retardation of the lakes is measured in months and years; and, unlike the retardation, which is a fraction of the cycle, the retardation in the river is the same absolute time, whether the flood-wave is long or short.

From these relations of the river it might be inferred that we are dealing with something like a limiting case of a chain of lakes, as, indeed, it is well known that we are. Were the Mississippi to run down to a zero of discharge, that would be exactly what it would show, only here their areas, some 5 to 15 miles long, and from a half to a mile wide, are almost lost sight of, in comparison with the great discharge capacity that exists between them. In general, also, it is a fact that at the highest stages the greatest velocities are found in the locations of the lakes themselves ; and with their functions thus reversed, they may be much more nearly taken to represent the points of outlets. It is not, however, a part of this study to consider theories of flow ; and it is enough here to note that, as the character of flow through a chain of lakes corresponds to observed facts in the river, it furnishes the basis of a radical departure from the whole series of assumptions upon which hydraulic formulas have heretofore rested.

TABLE No. 1—Water Levels of the Great Lakes.

[From the Report of the United States Deep Waterways Commission, 1896.]

Lake Superior at Superior, Wis., and Marquette, Mich.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 604.76 ft.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	3.85	4.45	4.03	3.53	3.00	2.88	2.81	2.75	2.65	2.72	2.79	3.53	3.25
1872...	3.85	3.96	4.08	4.18	3.53	3.15	2.88	2.71	2.55	2.65	2.80	3.10	3.29
1873...	3.20	3.65	3.62	3.61	3.01	2.71	2.42	2.24	2.18	2.28	2.42	2.72	2.84
1874...	3.17	3.19	3.23	3.13	3.06	2.86	2.48	2.39	2.29	2.23	2.41	2.72	2.76
1875...	3.04	3.08	3.04	3.04	2.82	2.46	2.47	2.38	2.15	2.30	2.44	2.64	2.65
1876...	2.84	3.05	3.14	3.11	2.57	1.89	1.50	1.39	1.50	1.83	1.99	2.27	2.26
1877...	2.63	2.87	3.13	3.21	3.22	3.00	2.62	2.56	2.72	2.72	2.93	3.00	2.88
1878...	3.12	3.00	3.77	3.80	3.53	3.25	3.18	3.30	3.47	3.40	3.60	3.92	3.45
1879...	3.82	3.86	3.56	3.95	4.31	4.08	3.84	3.72	3.83	3.74	3.82	4.12	3.89
1880...	4.44	4.48	4.57	4.54	3.85	3.02	2.87	2.88	2.88	2.93	2.99	3.25	3.56
1881...	3.51	3.61	3.70	3.79	3.49	3.05	2.99	2.94	2.71	2.37	2.44	2.72	3.11
1882...	3.07	3.32	3.43	3.51	3.35	3.33	2.88	2.76	2.72	2.89	2.91	3.10	3.11
1883...	3.33	3.62	3.62	3.37	3.36	3.26	3.01	2.99	3.03	3.23	3.38	3.49	3.31
1884...	3.56	3.59	3.71	4.00	3.78	3.58	3.44	3.43	3.16	2.80	2.90	3.11	3.42
1885...	3.34	3.52	3.60	3.65	3.32	3.04	2.80	2.68	2.75	2.92	3.07	3.40	3.17
1886...	3.60	3.73	3.79	3.70	3.45	3.31	3.24	3.33	3.35	3.25	3.40	3.54	3.47
1887...	3.85	3.83	3.52	3.35	3.56	3.40	3.12	3.04	3.18	3.25	3.49	3.71	3.44
1888...	3.82	3.81	3.88	3.88	3.41	2.63	2.44	2.30	2.35	2.44	2.58	2.93	3.04
1889...	3.25	3.47	3.64	3.63	3.28	3.16	2.97	2.78	2.65	2.81	3.12	3.42	3.18
1890...	3.56	3.69	3.93	3.96	3.75	3.30	3.00	2.85	2.72	2.75	2.96	3.32	3.32
1891...	3.68	3.81	3.85	3.89	3.69	3.64	3.44	3.37	3.43	3.38	3.49	3.90	3.63
1892...	3.90	4.18	4.31	4.30	3.97	3.59	3.56	3.44	3.39	3.49	3.66	3.94	3.81
1893...	4.22	4.31	4.26	4.16	3.66	3.14	2.84	2.78	2.87	2.90	3.06	3.29	3.46
1894...	3.47	3.65	3.56	3.41	2.63	2.41	2.35	2.22	2.30	2.28	2.33	2.52	2.76
1895...	2.82	3.04	3.21	3.31	2.94	2.62	2.42	2.37	2.23	2.18	2.47	2.80	2.70
Mean 25 yrs. 1871-95	3.48	3.63	3.69	3.68	3.38	3.07	2.86	2.78	2.76	2.79	2.94	3.22	3.19

TABLE No. 1—(*Continued*).

Lake Michigan, at Milwaukee, Wis.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 584.34 ft.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	3.12	3.20	2.60	2.40	2.05	2.01	1.98	2.21	2.88	3.57	3.62	4.21	2.82
1872...	4.34	4.34	4.56	4.31	4.06	3.69	3.66	3.68	3.75	3.87	4.16	4.82	4.10
1873...	4.82	4.78	4.47	3.90	3.34	2.71	2.75	2.65	2.84	2.90	3.13	3.17	3.45
1874...	3.21	2.92	2.77	2.87	2.89	2.52	2.59	2.58	2.83	3.18	3.38	3.72	2.96
1875...	3.92	3.99	3.93	3.57	3.01	2.77	2.80	2.63	2.70	2.85	3.06	3.25	3.21
1876...	3.30	3.10	2.77	2.57	1.95	1.54	1.20	1.27	1.32	1.90	1.80	2.27	2.08
1877...	2.41	2.40	2.40	2.02	2.13	2.06	2.09	2.21	2.42	2.41	2.53	2.59	2.31
1878...	2.71	2.78	2.62	2.60	2.30	2.16	2.15	2.47	2.67	2.78	2.91	3.23	2.62
1879...	3.54	3.53	3.49	3.50	3.37	3.30	3.21	3.40	3.52	3.74	3.96	3.93	3.54
1880...	3.89	3.98	3.94	3.77	3.43	2.89	2.67	2.64	2.94	3.28	3.60	3.77	3.40
1881...	3.76	3.55	3.26	2.35	2.84	2.61	2.64	2.64	2.87	2.54	2.71	2.81	2.88
1882...	3.03	3.04	2.67	2.54	2.44	2.17	2.11	1.92	2.04	2.45	2.66	2.99	2.51
1883...	3.25	3.21	3.13	2.91	2.43	2.07	1.47	1.50	1.69	1.91	2.36	2.44	2.36
1884...	2.66	2.54	2.29	2.11	1.90	1.74	1.90	2.04	2.29	2.29	2.65	2.68	2.26
1885...	2.67	2.44	2.48	2.29	1.93	1.72	1.63	1.42	1.56	1.70	2.00	2.29	2.01
1886...	2.06	2.04	1.76	1.49	1.23	1.16	1.35	1.58	1.82	1.92	2.26	2.59	1.77
1887...	2.67	2.30	2.14	2.19	1.99	1.86	1.92	2.06	2.40	2.85	3.18	3.30	2.41
1888...	3.48	3.53	3.35	3.14	2.76	2.49	2.48	2.60	2.75	3.00	3.05	3.63	3.02
1889...	3.65	3.68	3.70	3.69	3.61	3.15	2.97	3.21	3.38	3.63	3.98	4.16	3.57
1890...	4.08	4.12	4.14	3.82	3.59	3.18	3.11	3.19	3.39	3.50	3.84	4.19	3.68
1891...	4.21	4.45	4.26	3.95	3.85	3.70	3.87	3.94	4.17	4.53	4.93	4.99	4.24
1892...	4.87	4.68	4.78	4.72	4.30	3.85	3.84	3.76	3.96	4.20	4.47	4.74	4.35
1893...	4.75	4.61	4.50	4.04	3.74	3.41	3.39	3.56	3.88	4.02	4.41	4.48	4.07
1894...	4.47	4.44	4.18	4.03	3.49	3.33	3.30	3.38	3.81	4.02	4.29	4.64	3.95
1895...	4.82	4.93	4.96	4.76	4.60	4.55	4.66	4.78	5.05	5.42	5.64	5.75	4.99
Mean 25 yrs. 1871–95	3.59	3.54	3.41	3.18	2.93	2.67	2.63	2.68	2.92	3.14	3.39	3.63	3.14

TABLE No. 1—(*Continued*).

Lake Erie, at Cleveland, Ohio.

Monthly mean of water levels below the plane of reference of United States Lake Survey (high water of 1838).

Elevation of the plane of reference above mean tide at New York City is 575.20 ft.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	2.66	2.99	2.54	2.06	1.79	1.76	1.78	1.99	2.16	2.83	3.01	3.45	2.42
1872...	3.53	3.77	3.86	3.66	3.22	2.85	2.86	2.89	3.12	3.29	3.62	3.85	3.38
1873...	3.95	3.94	3.87	2.59	1.92	1.84	1.86	1.92	2.32	2.62	2.82	2.45	2.67
1874...	2.06	2.01	1.98	1.81	1.72	1.65	1.62	1.78	2.24	2.68	3.10	3.31	2.16
1875...	3.54	3.71	3.57	3.17	2.70	2.27	2.14	2.15	2.29	2.78	2.93	2.71	2.83
1876...	2.75	2.19	1.54	1.02	0.70	0.59	0.70	1.00	1.17	1.70	1.62	1.96	1.41
1877...	2.36	2.52	2.75	2.32	2.07	1.99	1.75	1.89	1.97	2.37	2.45	2.37	2.23
1878...	2.29	2.15	2.02	1.60	1.34	1.34	1.34	1.58	1.71	2.06	2.26	2.18	1.82
1879...	2.60	2.74	2.71	2.35	2.20	2.11	2.08	2.30	2.63	2.86	3.33	3.07	2.58
1880...	2.57	2.53	2.39	2.23	1.96	1.85	1.76	2.00	2.23	2.67	2.75	3.09	2.34
1881...	3.50	3.39	3.07	2.37	1.97	1.73	1.78	2.10	2.45	2.50	2.68	2.47	2.50
1882...	2.00	2.00	1.55	1.33	1.13	0.98	1.05	1.19	1.46	1.91	2.23	2.74	1.63
1883...	2.83	2.62	2.43	2.31	1.85	1.15	0.95	1.01	1.32	1.64	2.02	1.99	1.84
1884...	2.32	2.06	1.87	1.32	1.05	0.97	1.19	1.35	1.78	2.11	2.59	2.66	1.77
1885...	2.84	3.05	3.19	2.37	1.64	1.13	1.17	1.16	1.31	1.41	1.53	1.58	1.87
1886...	1.56	2.29	2.48	1.60	1.30	1.20	1.22	1.43	1.67	1.90	2.31	2.26	1.77
1887...	2.49	2.07	1.26	1.24	1.06	1.03	1.27	1.59	1.82	2.41	2.68	2.66	1.80
1888...	2.84	3.11	3.01	2.38	2.13	2.00	1.85	1.95	2.39	2.76	2.70	2.82	2.50
1889...	2.80	2.96	3.12	2.77	2.59	2.16	1.96	2.27	2.66	3.08	3.35	3.09	2.73
1890...	2.73	2.44	2.32	1.83	1.49	1.12	1.50	1.94	2.13	2.32	2.35	2.58	2.06
1891...	2.80	2.82	2.36	2.49	2.67	2.53	2.63	2.90	3.08	3.46	3.90	3.83	2.96
1892...	3.80	4.01	3.97	3.41	2.61	1.85	1.73	2.08	2.40	2.96	3.29	3.56	2.97
1893...	3.94	3.86	3.64	2.91	2.07	1.88	2.16	2.50	2.88	3.23	3.63	3.55	3.02
1894...	3.27	3.39	3.36	2.96	2.57	2.26	2.38	2.75	2.92	3.24	3.48	3.55	3.01
1895...	3.88	4.11	4.10	3.85	3.63	3.54	3.65	3.73	3.83	4.31	4.41	4.25	3.94
Mean 25 yrs. 1871–95	2.88	2.91	2.76	2.32	1.96	1.75	1.78	1.98	2.24	2.60	2.84	2.88	2.41

TABLE No. 1—(*Continued*).

Lake Ontario, at Oswego, New York.

Monthly mean of water levels above the zero of the United States Engineer gauge, the elevation of which zero is 244.21 ft. above mean tide at New York City.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Mean
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
1871...	1.94	1.77	1.98	2.58	3.00	2.94	2.78	2.34	2.00	1.50	1.09	0.78	2.06
1872...	0.61	0.39	0.23	0.72	0.84	1.17	1.23	1.07	0.78	0.62	0.57	0.23	0.71
1873...	0.19	0.26	0.38	2.34	2.88	2.80	2.76	2.48	2.06	1.61	1.48	1.67	1.74
1874...	2.23	2.63	3.18	3.07	3.05	3.14	3.09	2.86	2.22	1.82	1.25	0.91	2.45
1875...	0.61	0.26	0.52	1.32	1.59	1.75	1.78	1.64	1.43	1.15	0.96	0.78	1.15
1876...	1.19	1.85	2.40	3.38	3.96	4.18	4.25	3.79	3.18	2.84	2.48	2.30	2.98
1877...	1.77	1.50	1.65	2.34	2.41	2.31	2.35	2.08	1.65	1.22	1.13	1.26	1.81
1878...	1.36	1.57	2.27	2.52	2.86	2.85	2.81	2.73	2.47	2.21	2.09	2.90	2.39
1879...	2.69	2.33	2.18	2.59	2.68	2.71	2.55	2.20	1.78	1.34	0.95	0.98	2.08
1880...	1.20	1.48	1.82	2.00	2.15	2.39	2.40	1.97	1.60	1.19	1.15	0.98	1.69
1881...	0.62	0.61	1.27	1.69	1.87	2.09	2.16	1.84	1.28	1.06	1.06	1.06	1.38
1882...	1.61	1.78	2.38	2.71	2.90	3.41	3.40	3.07	2.69	2.18	1.76	1.47	2.45
1883...	1.20	1.26	1.50	2.02	2.67	3.37	3.90	3.72	3.24	2.80	2.57	2.43	2.56
1884...	2.39	2.76	3.44	4.05	4.07	3.97	3.76	3.53	3.10	2.68	2.18	2.03	3.16
1885...	2.02	1.75	1.47	2.15	2.95	3.32	3.46	3.31	3.09	2.90	2.95	3.12	2.71
1886...	3.48	3.55	3.69	4.31	4.52	4.32	3.92	3.48	3.12	2.83	2.39	2.30	3.49
1887...	2.05	2.80	3.31	3.52	4.08	4.04	3.76	3.25	2.64	2.25	1.90	1.63	2.94
1888...	1.32	1.18	1.42	2.05	2.12	2.16	2.22	2.12	1.73	1.37	1.30	1.29	1.69
1889...	1.50	1.64	1.81	2.05	2.20	2.51	2.70	2.45	1.89	1.45	1.05	1.62	1.91
1890...	2.13	2.48	2.81	3.05	3.41	4.04	3.87	3.20	2.85	2.52	2.60	2.39	2.95
1891...	2.07	2.33	2.87	3.35	3.13	2.71	2.43	1.99	1.56	0.92	0.32	0.29	2.00
1892...	0.39	0.36	0.49	1.07	1.13	1.69	2.20	2.12	1.92	1.48	1.21	1.08	1.26
1893...	0.75	0.64	1.12	1.87	3.03	3.25	2.99	2.45	2.18	1.66	1.25	1.10	1.86
1894...	1.44	1.62	1.92	1.97	2.15	2.68	2.48	1.90	1.39	1.14	0.81	0.46	1.66
1895...	0.37	0.31	0.21	0.76	0.88	0.76	0.47	+0.22	—0.12	—0.46	—0.71	—0.68	0.17
Mean 25 yrs. 1871–95	1.49	1.56	1.85	2.38	2.66	2.82	2.79	2.47	2.07	1.69	1.43	1.38	2.05

TABLE No. 2.—MONTHLY CHANGES IN WATER LEVEL OF THE GREAT
CUBIC FEET PER SECOND PER MONTH, AND THE CUM-

By cumulative effect is here meant the increase or decrease in the
level in the

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect, Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect, Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect, Superior—Michigan-Huron—Erie—Ontario.
1871.											
January	-0.09	- 30	-0.15	- 71	- 41	-0.21	- 22	+ 19	-0.07	- 5	+ 14
February	-0.60	-209	-0.08	- 40	-249	-0.33	- 36	-285	-0.17	- 14	-299
March	+0.42	+146	+0.60	+299	+445	+0.45	+ 49	+494	+0.21	- 17	+511
April	+0.50	+168	+0.20	+ 96	+264	+0.48	+ 51	+315	+0.60	+ 47	+362
May	+0.53	+178	+0.35	+169	+347	+0.27	+ 29	+376	+0.42	+ 33	+409
June	+0.12	+ 40	+0.04	+ 19	+ 59	+0.03	+ 3	+ 62	-0.06	- 5	+ 57
July	+0.07	+ 24	+0.03	+ 14	+ 38	-0.02	- 2	+ 36	-0.16	- 13	+ 23
August	-0.06	- 20	-0.23	-109	- 89	-0.21	- 22	-111	-0.44	- 34	-145
September	+0.10	+ 34	-0.67	-323	-289	-0.17	- 18	-307	-0.34	- 27	-334
October	-0.07	- 24	-0.69	-333	-357	0.67	- 71	-428	0.50	- 39	-467
November	-0.07	- 24	-0.05	- 24	- 48	-0.18	- 19	- 67	-0.41	- 32	- 99
December	-0.74	-249	-0.59	-285	-534	-0.44	- 47	-581	-0.31	- 24	-605
1872.											
January	-0.32	-106	-0.13	- 62	-168	-0.08	- 8	-176	-0.17	- 13	-189
February	-0.11	- 38	0.00	00	- 38	-0.24	- 26	- 64	-0.22	- 17	- 81
March	-0.12	- 41	-0.22	-108	-149	-0.09	- 10	-159	-0.16	- 13	-172
April	+0.10	+ 34	-0.25	-121	- 87	+0.20	+ 21	+108	+0.49	+ 39	+147
May	+0.65	+219	+0.25	+121	+340	+0.44	+ 47	+387	+0.12	+ 9	+396
June	+0.38	+128	+0.37	+178	+306	+0.37	+ 39	+345	+0.33	+ 26	+371
July	+0.27	+ 91	+0.03	+ 14	+105	-0.01	- 1	+104	+0.06	+ 5	+109
August	+0.17	+ 56	-0.02	- 9	+ 47	-0.03	- 3	+ 44	-0.16	- 12	+ 32
September	-0.16	- 54	-0.07	- 34	- 20	-0.23	- 24	- 4	-0.29	- 23	- 27
October	-0.10	- 34	-0.12	- 58	- 92	-0.17	- 18	-110	-0.16	- 13	-123
November	-0.15	- 50	-0.29	-140	-190	-0.33	- 35	-225	-0.05	- 4	-229
December	-0.30	-101	-0.66	-318	-419	-0.23	- 24	-443	-0.34	- 27	-470
1873.											
January	-0.10	- 33	0.00	000	- 33	-0.10	- 10	- 43	-0.04	- 3	- 46
February	-0.45	-156	-0.04	- 20	-136	-0.01	- 1	-135	+0.07	+ 6	-129
March	+0.03	+ 10	+0.31	+155	+165	+0.07	+ 8	+173	+0.12	+ 10	+183
April	+0.01	+ 3	+0.57	+275	+278	+1.28	+135	+413	+1.96	+154	+567
May	+0.60	+202	+0.56	+270	+472	+0.67	+ 71	+543	+0.54	+ 43	+586
June	+0.30	+101	+0.63	+304	+405	+0.08	+ 8	+413	-0.08	- 6	+407
July	+0.29	+ 98	-0.04	- 19	+ 79	-0.02	- 2	+ 77	-0.04	- 3	+ 74
August	+0.18	+ 60	+0.10	+ 47	+107	-0.06	- 6	+101	-0.28	- 22	+ 79
September	+0.06	+ 20	-0.19	- 92	- 72	-0.40	- 42	-114	-0.42	- 33	-147
October	-0.10	- 34	-0.06	- 29	- 63	-0.30	- 32	- 95	-0.45	- 35	-130
November	-0.14	- 47	-0.23	-111	-158	-0.20	- 21	-179	-0.13	- 10	-189
December	-0.30	-101	-0.04	- 19	-120	+0.37	+ 39	- 81	+0.19	+ 15	- 66
1874.											
January	-0.45	-149	0.04	- 19	-168	+0.39	+ 41	-127	+0.56	+ 43	- 84
February	-0.02	- 7	+0.29	+145	+138	+0.05	+ 5	+143	+0.40	+ 33	+176
March	-0.04	- 14	+0.15	+ 75	+ 61	+0.03	+ 3	+ 64	+0.55	+ 45	+109
April	+0.10	+ 34	-0.10	- 48	- 14	+0.17	+ 18	+ 4	-0.11	- 9	- 5
May	+0.07	+ 24	-0.02	- 10	+ 14	+0.09	+ 10	+ 24	-0.02	- 2	+ 22
June	+0.20	+ 67	+0.37	+178	+245	+0.07	+ 7	+252	+0.09	+ 7	+259
July	+0.38	+128	-0.07	- 34	+ 94	+0.03	+ 3	+ 97	-0.05	- 4	+ 98
August	+0.09	+ 30	+0.01	+ 5	+ 35	-0.16	- 17	+ 18	-0.23	- 18	000
September	+0.10	+ 34	-0.25	-121	- 87	-0.46	- 49	-136	-0.64	- 50	-186
October	+0.06	+ 20	-0.35	-169	-149	-0.44	- 47	-196	-0.40	- 32	-228
November	-0.18	- 61	-0.20	- 96	-157	-0.42	- 44	-201	-0.57	- 45	-246
December	-0.31	-104	0.34	-164	-268	-0.21	- 22	-290	-0.34	- 27	-317

LAKES, WITH CORRESPONDING RATES OF STORAGE IN THOUSANDS OF
ULATIVE EFFECTS IN THE LAKES BELOW SUPERIOR.

normal discharge of any outlet by the elimination of oscillations of lakes above.

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan—Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan—Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan—Huron—Erie—Ontario.
1875.											
January	—0.32	—106	—0.20	— 95	—201	—0.23	— 24	—225	—0.30	— 23	—248
February	—0.04	— 14	—0.07	— 35	— 49	—0.17	— 19	— 68	—0.35	29	— 97
March.........	+0.04	+ 14	+0.06	+ 30	+ 44	+0.14	+ 15	+ 59	+0.26	+ 21	+ 80
April..........	0.00	......	+0.36	+174	+174	+0.40	+ 42	+216	+0.80	+ 63	+279
May...........	+0.22	+ 74	+0.56	+270	+344	+0.47	+ 50	+394	+0.27	+ 21	+415
June..........	+0.36	+121	+0.24	+116	+237	+0.43	+ 45	+282	+0.16	+ 13	+295
July	—0.01	— 3	0.03	— 14	— 17	+0.13	+ 14	— 3	+0.03	+ 2	— 1
August	+0.09	+ 30	+0.17	+ 81	+111	—0.01	— 1	+110	—0.14	— 11	+ 99
September....	+0.23	+ 77	—0.07	— 34	+ 43	—0.14	— 15	+ 28	—0.21	— 17	+ 11
October.......	—0.15	— 50	—0.15	— 72	—122	—0.49	— 52	—174	—0.28	— 22	—196
November	—0.14	— 47	—0.21	—101	—148	—0.15	— 16	—164	—0.19	— 15	—179
December.....	—0.20	— 67	—0.19	— 92	—159	+0.22	+ 23	—136	—0.18	— 14	—150
1876.											
January	—0.20	— 66	—0.05	— 24	— 90	—0.04	— 4	— 94	+0.41	+ 32	— 62
February	—0.21	— 72	+0.20	+ 98	+ 26	+0.56	+ 60	+ 86	+0.66	+ 53	+139
March.........	—0.09	— 30	+0.33	+162	+132	+0.65	+ 70	+202	+0.55	+ 44	+246
April..........	+0.03	+ 10	+0.20	+ 96	+106	+0.52	+ 55	+161	+0.98	+ 77	+238
May...........	+0.54	+182	+0.62	+299	+481	+0.32	+ 34	+515	+0.58	+ 46	+561
June..........	+0.68	+229	+0.41	+198	+427	+0.11	+ 12	+439	+0.22	+ 17	+456
July	+0.39	+131	+0.34	+164	+295	—0.11	— 12	+283	+0.07	+ 6	+289
August........	+0.11	+ 36	—0.07	— 33	+ 3	—0.30	— 31	— 28	—0.46	— 36	— 64
September....	—0.11	— 37	—0.05	— 24	— 61	—0.17	— 18	— 79	—0.61	— 48	—127
October.......	—0.33	—111	—0.58	—280	—391	—0.53	— 56	—447	—0.34	— 27	—474
November	—0.16	— 54	+0.10	+ 48	— 6	+0.08	+ 8	+ 2	—0.36	— 28	— 26
December.....	—0.28	— 94	—0.47	—227	—321	—0.34	— 36	—357	—0.18	— 14	—371
1877.											
January	—0.36	—119	—0.14	— 66	—185	—0.40	— 42	—227	—0.53	— 40	—267
February	—0.24	— 83	+0.01	+ 5	— 78	—0.16	— 17	— 95	—0.27	— 22	—117
March.........	—0.26	— 90	0.00	......	— 90	—0.23	— 25	—115	+0.15	+ 12	—103
April..........	—0.08	— 27	+0.38	+183	+156	+0.43	+ 45	+201	+0.69	+ 54	+255
May...........	—0.01	3	—0.11	— 53	— 56	+0.25	+ 26	— 30	+0.07	+ 6	— 24
June..........	+0.22	+ 74	+0.07	+ 34	+108	+0.08	+ 8	+116	—0.10	— 8	+108
July	+0.38	+128	—0.03	— 14	+114	+0.24	+ 25	+139	+0.04	+ 3	+142
August	+0.06	+ 20	—0.12	— 57	— 37	—0.14	— 15	— 52	—0.27	— 21	— 73
September....	—0.16	— 54	—0.21	—101	—155	—0.08	— 8	—163	—0.43	— 34	—197
October.......	0.00	......	+0.01	+ 5	+ 5	—0.40	— 42	— 37	—0.43	— 34	— 71
November	—0.21	— 71	—0.12	— 58	—129	—0.08	— 8	—137	—0.09	— 7	—144
December.....	—0.07	— 24	—0.06	— 29	— 53	+0.08	+ 8	— 45	+0.13	+ 10	— 35
1878.											
January	—0.12	— 40	—0.12	— 57	— 97	+0.08	+ 8	— 89	+0.10	+ 8	— 81
February	+0.12	+ 42	0.07	— 35	+ 7	+0.14	+ 15	+ 22	+0.21	+ 17	+ 39
March.........	—0.77	—268	+0.16	+ 80	—188	+0.13	+ 14	—174	+0.70	+ 57	—117
April..........	—0.03	— 10	+0.02	+ 10	00	+0.42	+ 44	+ 44	+0.25	+ 20	+ 64
May...........	+0.27	+ 91	+0.30	+145	+236	+0.26	+ 27	+263	+0.34	+ 27	+290
June..........	+0.28	+ 94	+0.14	+ 68	+162	0.00	......	+162	—0.01	— 1	+161
July	+0.07	+ 24	+0.01	+ 5	+ 29	0.00	......	+ 29	—0.04	— 3	+ 26
August	—0.12	— 40	—0.32	—152	—192	—0.24	— 25	—217	—0.08	— 6	—223
September....	—0.17	— 57	—0.20	— 96	—153	—0.13	— 14	—167	—0.26	— 20	—187
October.......	+0.07	+ 24	—0.11	— 53	— 29	—0.35	— 37	— 66	—0.26	— 20	— 86
November	—0.20	— 67	—0.13	— 63	—130	—0.20	— 21	—151	—0.12	— 9	—160
December.....	—0.32	—104	—0.32	—159	—263	+0.08	+ 8	—255	+0.81	+ 64	—191

TABLE No. 2—

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect,—Superior—Michigan-Huron—Erie—Ontario.
1879.											
January	+0.10	+ 33	—0.31	—147	—114	—0.42	— 44	—158	—0.21	— 16	—174
February	—0.04	— 14	+0.01	+ 5	— 9	—0.14	— 15	— 24	—0.36	— 29	— 53
March	+0.30	+104	+0.04	+ 20	+124	+0.03	+ 3	+127	—0.15	— 12	+115
April	—0.39	—131	—0.01	— 5	—136	+0.36	+ 38	— 98	+0.41	+ 32	— 66
May	—0.36	—121	+0.13	+ 63	— 58	+0.15	+ 16	— 42	+0.09	+ 7	— 35
June	+0.23	+ 77	+0.07	+ 34	+111	+0.09	+ 10	+121	+0.03	+ 2	+123
July	+0.24	+ 81	+0.09	+ 43	+124	+0.03	+ 3	+127	—0.16	— 13	+114
August	+0.12	+ 40	—0.19	— 90	— 50	—0.22	— 23	— 73	—0.35	— 27	—100
September	—0.11	— 37	—0.12	— 58	— 95	—0.33	— 35	—130	—0.42	— 33	—163
October	+0.09	+ 30	—0.22	—106	— 76	—0.23	— 24	—100	—0.44	— 35	—135
November	—0.08	— 27	—0.22	—106	—133	—0.47	— 50	—183	—0.39	— 31	—214
December	—0.30	—101	+0.03	+ 14	— 87	+0.26	+ 27	— 60	+0.03	+ 2	— 58
1880.											
January	—0.32	—106	+0.04	+ 19	— 87	+0.50	+ 52	— 35	+0.22	+ 17	— 18
February	—0.04	— 14	—0.09	— 44	— 58	+0.04	+ 4	— 54	+0.28	+ 22	— 32
March	—0.09	— 31	+0.04	+ 20	— 11	+0.14	+ 15	+ 4	+0.34	+ 27	+ 31
April	+0.03	+ 10	+0.17	+ 82	+ 92	+0.16	+ 17	+109	+0.18	+ 14	+123
May	+0.69	+232	+0.34	+164	+396	+0.27	+ 29	+425	+0.15	+ 12	+437
June	+0.83	+279	+0.54	+260	+539	+0.11	+ 12	+551	+0.24	+ 19	+570
July	+0.15	+ 50	+0.22	+106	+156	+0.09	+ 10	+166	+0.01	+ 1	+167
August	—0.01	— 3	+0.03	+ 14	+ 11	—0.24	— 25	— 14	—0.43	— 33	— 47
September	0.00	......	—0.30	—145	—145	—0.23	— 24	—169	—0.37	— 29	—198
October	—0.05	— 17	—0.34	—164	—181	—0.44	— 47	—228	—0.41	— 32	—260
November	—0.06	— 20	—0.32	—154	—174	—0.08	— 8	—182	—0.14	— 11	—193
December	—0.26	— 87	—0.17	— 82	—169	—0.34	— 36	—205	—0.17	— 13	—218
1881.											
January	—0.26	— 86	+0.01	+ 5	— 81	—0.41	— 43	—124	—0.36	— 28	—152
February	—0.10	— 35	+0.21	+105	+ 70	+0.11	+ 12	+ 82	—0.01	— 1	+ 81
March	—0.09	— 31	+0.29	+145	+114	+0.32	+ 35	+149	+0.66	+ 54	+203
April	—0.09	— 30	—0.09	— 43	— 73	+0.70	+ 74	+ 1	+0.42	+ 33	+ 34
May	+0.30	+101	—0.49	—236	—135	+0.40	+ 42	— 93	+0.18	+ 14	— 79
June	+0.44	+148	+0.23	—111	+259	—0.24	— 25	+284	+0.22	+ 17	+301
July	+0.06	+ 20	—0.03	— 14	+ 6	—0.05	— 5	+ 1	+0.07	+ 6	+ 7
August	+0.05	+ 17	0.00	......	+ 17	—0.32	— 33	— 16	—0.32	— 25	— 41
September	+0.23	+ 77	—0.23	—111	— 34	—0.35	— 37	— 71	—0.56	— 44	—115
October	+0.34	+114	+0.33	+159	+273	—0.05	— 5	+268	—0.22	— 17	+251
November	—0.07	— 24	—0.17	— 82	—106	—0.18	— 19	—125	0.00	......	—125
December	—0.28	— 94	—0.10	— 48	—142	+0.21	+ 22	—120	0.00	......	—120
1882.											
January	—0.35	—116	—0.22	—104	—220	+0.47	+ 49	—171	+0.55	+ 43	—128
February	—0.25	— 87	—0.01	— 5	— 92	0.00	......	— 92	+0.17	+ 14	— 78
March	—0.11	— 38	+0.37	+184	+146	+0.45	+ 49	+195	+0.60	+ 49	+244
April	—0.08	— 27	+0.13	+ 63	+ 36	+0.22	+ 23	+ 59	+0.33	+ 26	+ 85
May	+0.16	+ 54	+0.10	+ 48	+102	+0.20	+ 21	+123	+0.19	+ 15	+138
June	+0.02	+ 7	+0.27	+130	+137	+0.15	+ 16	+153	+0.51	+ 40	+193
July	+0.45	+151	+0.06	+ 29	+180	—0.07	— 7	+173	—0.01	— 1	+172
August	+0.12	+ 40	+0.19	+ 90	+130	—0.14	— 15	+115	—0.33	— 26	+ 89
September	+0.04	+ 13	—0.12	— 58	— 45	—0.27	— 29	— 74	—0.38	— 30	—104
October	—0.17	— 57	—0.41	—198	—255	—0.45	— 48	—303	—0.51	— 40	—343
November	—0.02	— 7	—0.21	—101	—108	—0.32	— 34	—142	—0.42	— 33	—175
December	—0.19	— 64	—0.33	—159	—223	—0.51	— 54	—277	—0.29	— 23	—300
1883.											
January	—0.23	— 76	—0.26	—123	—199	—0.09	— 9	—208	—0.27	— 21	—229
February	—0.29	—101	+0.04	+ 20	— 81	+0.21	+ 23	— 58	+0.06	+ 5	— 53
March	0.00	.....	+0.09	+ 45	+ 45	+0.19	+ 21	+ 66	+0.24	+ 20	+ 86
April	+0.25	+ 84	+0.21	+101	+185	+0.12	+ 13	+198	+0.52	+ 41	+239
May	+0.01	+ 3	+0.48	+231	+234	+0.46	+ 49	+283	+0.65	+ 51	+334
June	+0.10	+ 34	+0.36	+174	+208	+0.70	+ 74	+282	+0.70	+ 55	+337
July	+0.25	+ 84	+0.60	+289	+373	+0.20	+ 21	+394	+0.53	+ 42	+436
August	+0.02	+ 7	—0.03	— 14	— 7	—0.06	— 6	— 13	—0.18	— 14	— 27
September	—0.04	— 13	—0.19	— 92	—105	—0.31	— 33	—138	—0.48	— 38	—176
October	—0.20	— 67	—0.22	—106	—173	—0.32	— 34	—207	—0.44	— 35	—242
November	—0.15	— 50	—0.45	—217	—267	—0.38	— 40	—307	—0.23	— 18	—325
December	—0.11	— 37	—0.08	— 39	— 76	+0.03	+ 3	— 73	—0.14	— 11	— 84

(*Continued*).

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fuctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1884.											
January	-0.07	-23	-0.22	-104	-127	-0.33	-34	-161	-0.04	-3	-164
February	-0.03	-10	+0.12	+59	+49	+0.26	+28	+77	+0.37	+30	+107
March	-0.12	-41	+0.25	+123	+82	+0.19	+20	+102	+0.68	+54	+156
April	-0.29	-98	+0.18	+87	-11	+0.55	+58	+47	+0.61	+48	+95
May	+0.22	+74	+0.21	+101	+175	+0.27	+29	+204	+0.02	+2	+206
June	+0.20	+67	+0.16	+77	+144	+0.08	+8	+152	-0.10	-8	+144
July	+0.14	+47	-0.16	-77	-30	-0.22	-23	-53	-0.21	-17	-70
August	+0.01	+3	-0.14	-66	-63	-0.16	-17	-80	-0.23	-18	-98
September	+0.27	+91	-0.25	-121	-30	-0.43	-45	-75	-0.43	-34	-109
October	+0.36	+121	0.00	.....	+121	-0.33	-35	+86	-0.42	-33	+53
November	-0.10	-34	-0.36	-174	-208	-0.48	-51	-259	-0.50	-39	-298
December	-0.21	-71	-0.03	-14	-85	-0.07	-7	-92	-0.15	-12	-104
1885.											
January	-0.23	-76	+0.01	+5	-71	-0.18	-19	-90	-0.01	-1	-91
February	-0.18	-63	+0.23	+115	+52	-0.21	-23	+29	-0.27	-22	+7
March	-0.08	-28	-0.04	-20	-48	-0.14	-15	-63	-0.28	-23	-86
April	-0.05	-17	+0.19	+92	+75	+0.82	+87	+162	+0.68	+54	+216
May	+0.33	+111	+0.36	+174	+285	+0.73	+77	+362	+0.80	+63	+425
June	+0.28	+94	+0.21	+101	+195	+0.51	+54	+249	+0.37	+29	+278
July	+0.24	+81	+0.09	+43	+124	-0.04	-4	+120	+0.14	+11	+131
August	+0.12	+40	+0.21	+100	+140	+0.01	+1	+141	-0.15	-12	+129
September	-0.07	-24	-0.14	-68	-92	-0.15	-16	-108	-0.22	-17	-125
October	-0.17	-57	-0.14	-68	-125	-0.10	-11	-136	-0.19	-15	-151
November	-0.15	-50	-0.30	-145	-195	-0.12	-13	-208	+0.05	+4	-204
December	-0.33	111	-0.29	-140	-251	-0.05	-5	-256	+0.17	+13	-243
1886.											
January	-0.20	-66	+0.23	+109	+43	+0.02	+2	+45	+0.36	+28	+73
February	-0.13	-45	+0.02	+10	-35	-0.73	-80	-115	+0.07	+6	-109
March	-0.06	-21	+0.28	+140	+119	-0.19	-21	+98	+0.14	+11	+109
April	+0.09	+30	+0.27	+130	+160	+0.88	+93	+253	+0.62	+49	+302
May	+0.25	+84	+0.26	+125	+209	+0.30	+32	+241	+0.21	+17	+258
June	+0.14	+47	+0.07	+34	+81	+0.10	+11	+92	-0.20	-16	+76
July	+0.07	+24	-0.19	92	-68	-0.02	-2	-70	-0.40	-32	-102
August	-0.09	-30	-0.23	-109	-139	-0.21	-22	-161	-0.44	-34	-195
September	-0.02	-7	-0.24	-116	-123	-0.24	-25	-148	-0.36	-28	-176
October	+0.10	+34	-0.10	-48	-14	-0.23	-24	-38	-0.29	-23	-61
November	-0.15	-50	-0.34	-164	-214	-0.41	-43	-257	-0.44	-35	-292
December	-0.14	-47	-0.33	-159	-206	+0.05	+5	-201	-0.09	-7	-208
1887.											
January	-0.31	-103	-0.08	-38	-141	-0.23	-24	-165	-0.25	-19	-184
February	+0.02	+7	+0.37	+184	+191	+0.42	+46	+237	+0.75	+61	+298
March	+0.31	+108	+0.16	+80	+188	+0.81	+89	+277	+0.51	+42	+319
April	+0.17	+57	-0.05	-24	+33	+0.02	+2	+35	+0.21	+17	+52
May	-0.21	-71	+0.20	+96	+25	+0.18	+19	+44	+0.56	+44	+88
June	+0.16	+54	+0.13	+63	+117	+0.03	+3	+120	-0.04	-3	+117
July	+0.28	+94	-0.06	29	+65	-0.24	-25	+40	-0.28	-22	+18
August	+0.08	+26	-0.14	-66	-40	-0.32	-33	-73	-0.51	-40	-113
September	-0.14	-47	-0.34	-164	211	-0.23	-24	-235	-0.61	-48	-283
October	-0.07	-24	-0.45	-217	-241	-0.59	-62	-303	-0.39	-31	-334
November	-0.24	-81	-0.33	-159	-240	-0.27	-29	-269	-0.35	-27	-296
December	-0.22	-74	0.12	-58	-132	+0.02	+2	-130	-0.27	-21	-151
1888.											
January	-0.11	-36	-0.18	-85	-121	-0.18	-19	-140	-0.31	-24	-164
February	+0.01	+3	-0.05	-25	-22	-0.27	-29	-51	-0.14	-11	-62
March	-0.07	-24	+0.18	+88	+64	+0.10	+11	+75	+0.24	+19	+94
April	0.00	.....	+0.21	+101	+101	+0.63	+67	+168	+0.63	+50	+218
May	+0.47	+158	+0.38	+183	+341	+0.25	+26	+367	+0.07	+6	+373
June	+0.78	+262	+0.27	+130	+392	+0.13	+14	+406	+0.04	+3	+409
July	+0.19	+64	+0.01	+5	+69	+0.15	+16	+85	+0.06	+5	+90
August	+0.14	+46	-0.12	-57	-11	-0.10	-10	-21	-0.10	-8	-29
September	-0.05	-17	-0.15	-72	-89	-0.44	-47	-136	-0.39	-31	-167
October	-0.09	-30	-0.25	-121	-151	-0.37	-39	-190	-0.36	-28	-218
November	-0.14	-47	-0.05	-24	-71	+0.06	+6	-65	-0.07	-6	-71
December	0.35	-118	0.58	-280	398	-0.12	-13	-412	-0.01	-1	-413

TABLE No. 2—

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1889.											
January......	—0.32	—106	—0.02	— 9	—115	+0.02	+ 2	—113	+0.21	+ 16	— 97
February.....	—0.22	— 77	—0.03	— 15	— 92	—0.16	— 17	—109	+0.14	+ 11	— 98
March.........	—0.17	— 59	—0.02	— 10	— 69	—0.16	— 17	— 86	+0.17	+ 14	— 72
April..........	+0.01	+ 3	+0.01	+ 5	+ 8	+0.35	+ 37	+ 45	+0.24	+ 19	+ 64
May...........	+0.35	+118	+0.08	+ 39	+157	+0.18	+ 19	+176	+0.15	+ 12	+188
June..........	+0.12	+ 40	+0.46	+222	+262	+0.43	+ 45	+307	+0.31	+ 24	+331
July..........	+0.19	+ 64	+0.18	+ 87	+151	+0.20	+ 21	+172	+0.19	+ 15	+187
August.......	+0.19	+ 63	—0.24	—114	— 51	—0.31	— 32	— 83	—0.25	— 19	—102
September....	+0.13	+ 44	—0.17	— 82	— 38	—0.39	— 41	— 79	—0.56	— 44	—123
October......	—0.16	— 54	—0.25	—121	—175	—0.42	— 44	—219	—0.44	— 35	—254
November....	—0.31	—104	—0.35	—169	—273	—0.27	— 29	—302	—0.40	— 32	—334
December.....	—0.30	—101	—0.18	— 87	—188	+0.26	+ 27	—161	+0.57	+ 45	—116
1890.											
January......	—0.14	— 46	+0.08	+ 38	— 8	+0.36	+ 37	+ 29	+0.51	+ 40	+ 69
February.....	—0.13	— 45	—0.04	— 20	— 65	+0.29	+ 32	— 33	+0.35	+ 29	— 4
March.........	—0.24	— 83	—0.02	— 10	— 93	+0.12	+ 13	— 80	+0.33	+ 27	— 53
April..........	—0.03	— 10	+0.32	+154	+144	+0.49	+ 52	+196	+0.24	+ 19	+215
May...........	+0.21	+ 71	+0.23	+111	+182	+0.34	+ 36	+218	+0.36	+ 28	+246
June..........	+0.45	+151	+0.41	+198	+349	+0.37	+ 39	+388	+0.63	+ 50	+438
July..........	+0.30	+101	+0.07	+ 34	+135	—0.38	— 40	+ 95	—0.17	— 13	+ 82
August.......	+0.15	+ 50	—0.08	— 38	+ 12	—0.44	— 46	— 34	—0.67	— 52	— 86
September....	+0.13	+ 44	—0.20	— 96	— 52	—0.19	— 20	— 72	—0.35	— 28	—100
October......	—0.03	— 10	—0.11	— 53	— 63	—0.19	— 20	— 83	—0.33	— 26	—109
November....	—0.21	— 71	—0.34	—164	—235	—0.03	— 3	—238	+0.08	+ 6	—232
December.....	—0.36	—121	—0.35	—169	—290	—0.23	— 24	—314	—0.21	— 17	—331
1891.											
January......	—0.36	—119	—0.02	— 9	—128	—0.22	— 23	—151	—0.32	— 25	—176
February.....	—0.13	— 45	—0.24	—120	—165	—0.02	— 2	—167	+0.26	+ 21	—146
March.........	—0.04	— 14	+0.19	+ 95	+ 81	+0.46	+ 50	+131	+0.54	+ 44	+175
April..........	—0.04	— 13	+0.31	+150	+137	—0.13	— 14	+123	+0.48	+ 38	+161
May...........	+0.20	+ 67	+0.10	+ 48	+115	—0.18	— 19	+ 96	—0.22	— 17	+ 79
June..........	+0.05	+ 17	+0.15	+ 72	+ 89	+0.14	+ 15	+104	—0.42	— 33	+ 71
July..........	+0.20	+ 67	—0.17	— 82	— 15	—0.10	— 11	— 26	—0.28	— 22	— 48
August.......	+0.07	+ 23	—0.07	— 33	— 10	—0.27	— 28	— 38	—0.44	— 34	— 72
September....	—0.06	— 20	—0.23	—111	—131	—0.18	— 19	—150	—0.43	— 34	—184
October......	+0.05	+ 17	—0.36	—174	—157	—0.38	— 40	—197	—0.64	— 50	—247
November....	—0.11	— 37	—0.40	—193	—230	—0.44	— 47	—277	—0.60	— 47	—324
December.....	—0.41	—139	—0.06	— 29	—168	+0.07	+ 7	—161	—0.03	— 2	—163
1892.											
January......	0.00	.....	+0.12	+ 57	+ 57	+0.03	+ 3	+ 60	+0.10	+ 8	+ 68
February.....	—0.28	— 96	+0.19	+ 93	— 3	—0.21	— 23	— 26	—0.03	— 2	— 28
March.........	—0.13	— 44	—0.10	— 49	— 93	+0.04	+ 4	— 89	+0.13	+ 10	— 79
April..........	+0.01	+ 3	+0.06	+ 29	+ 32	+0.56	+ 58	+ 90	+0.58	+ 46	+136
May...........	+0.33	+111	+0.42	+203	+314	+0.80	+ 85	+399	+0.06	+ 5	+404
June..........	+0.38	+128	+0.45	+217	+345	+0.76	+ 80	+425	+0.56	+ 44	+469
July..........	+0.03	+ 10	+0.01	+ 5	+ 15	+0.12	+ 13	+ 28	+0.51	+ 40	+ 68
August.......	+0.12	+ 40	+0.08	+ 38	+ 78	—0.35	— 36	+ 42	—0.08	— 6	+ 36
September...	+0.05	+ 17	—0.20	— 96	— 79	—0.32	— 34	—113	—0.20	— 16	—129
October.......	—0.10	— 34	—0.24	—116	—150	—0.56	— 59	—209	—0.44	— 35	—244
November....	—0.17	— 57	—0.27	—130	—187	—0.33	— 35	—222	—0.27	— 21	—243
December.....	—0.28	— 94	—0.27	—130	—224	—0.27	— 29	—253	—0.13	— 10	—263

(*Continued*).

	Superior.		Michigan-Huron.			Erie.			Ontario.		
	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie.	Fluctuation in feet.	Storage in 1 000 second-feet per month.	Cumulative effect.—Superior—Michigan-Huron—Erie—Ontario.
1893.											
January	−0.28	− 93	−0.01	− 5	− 98	−0.38	− 40	−138	−0.33	− 26	−164
February	−0.09	− 31	+0.14	+ 70	+ 39	+0.08	+ 9	+ 48	−0.11	− 9	+ 39
March..........	+0.05	+ 17	+0.11	+ 55	+ 72	+0.22	+ 24	+ 96	+0.48	+ 39	+135
April...........	+0.10	+ 34	+0.46	+222	+256	+0.73	+ 77	+333	+0.75	+ 59	+392
May	+0.50	+168	+0.30	+145	+313	+0.84	+ 89	+402	+1.16	+ 91	+493
June	+0.52	+175	+0.33	+159	+334	+0.19	+ 20	+354	+0.22	+ 17	+371
July	+0.30	+101	+0.02	+ 10	+111	−0.28	− 30	+ 81	−0.26	− 20	+ 61
August	+0.06	+ 20	−0.17	− 81	− 61	−0.34	− 35	− 96	−0.54	− 42	−138
September....	−0.09	− 30	−0.32	−154	−184	−0.38	− 40	−224	−0.27	− 21	−245
October	−0.03	− 10	−0.14	− 68	− 78	−0.35	− 37	−115	−0.52	− 41	−156
November	−0.16	− 54	−0.39	−188	−242	−0.40	− 42	−284	−0.41	− 32	−316
December.....	−0.23	− 77	−0.07	− 34	−111	+0.08	+ 8	−103	−0.15	− 12	−115
1894.											
January	−0.18	− 60	+0.01	+ 5	− 55	+0.28	+ 29	− 26	+0.34	+ 26	0
February	−0.18	− 63	+0.03	+ 15	− 48	−0.12	− 13	− 61	+0.18	+ 15	− 46
March..........	+0.09	+ 31	+0.26	+130	+161	+0.03	+ 3	+164	+0.30	+ 24	+188
April...........	+0.15	+ 50	+0.15	+ 72	+122	+0.40	+ 42	+164	+0.05	+ 4	+168
May............	+0.78	+262	+0.54	+260	+522	+0.39	+ 41	+563	+0.18	+ 14	+577
June	+0.22	+ 74	+0.16	+ 77	+151	+0.31	+ 33	+184	+0.53	+ 42	+226
July	+0.06	+ 20	+0.03	+ 14	+ 34	−0.12	− 13	+ 21	−0.20	− 16	+ 5
August	+0.13	+ 43	−0.08	− 38	+ 5	−0.37	− 39	− 34	−0.58	− 45	− 79
September....	−0.08	− 27	−0.43	−207	−234	−0.17	− 18	−252	−0.51	− 40	−292
October	+0.02	+ 7	−0.21	−101	− 94	−0.32	− 34	−128	−0.25	− 20	−148
November	−0.05	− 17	−0.27	−130	−147	−0.24	− 25	−172	−0.33	− 26	−198
December.....	−0.19	− 64	−0.35	−169	−233	−0.07	− 7	−240	−0.35	− 28	−268
1895.											
January	−0.30	− 99	−0.18	− 85	−184	−0.33	− 34	−218	−0.09	− 7	−225
February	−0.22	− 77	−0.11	− 55	−132	−0.23	− 25	−157	−0.06	− 5	−162
March	−0.17	− 59	−0.03	− 15	− 74	+0.01	+ 1	− 73	−0.10	− 8	− 81
April..........	−0.10	− 34	+0.20	+ 96	+ 62	+0.25	+ 26	+ 88	+0.55	+ 43	+131
May............	+0.37	+124	+0.16	+ 77	+201	+0.22	+ 23	+224	+0.12	+ 9	+233
June...........	+0.32	+108	+0.05	+ 24	+132	+0.09	+ 10	+142	−0.12	− 9	+133
July	+0.20	+ 67	−0.11	− 53	+ 14	−0.11	− 12	+ 2	−0.29	− 23	− 21
August	+0.05	+ 17	−0.12	− 57	− 40	−0.08	− 8	− 48	−0.25	− 19	− 67
September....	+0.14	+ 47	−0.27	−130	− 83	−0.10	− 11	− 94	−0.34	− 27	−121
October	+0.05	+ 17	−0.37	−178	−161	−0.48	− 51	−212	−0.34	− 27	−239
November	−0.29	− 98	−0.22	−106	−204	−0.10	− 11	−215	−0.25	− 20	−235
December.....	−0.33	−111	−0.11	− 53	−164	+0.16	+ 17	−147	+0.03	+ 2	−145
Mean for 25 years. 1871–95.											
January	−0.26	− 86	−0.04	− 19	−105	0.00	0	−105	+0.11	+ 8	− 97
February	−0.15	− 52	+0.05	+ 25	− 27	−0.03	− 3	− 30	+0.07	+ 6	− 24
March..........	−0.06	− 21	+0.13	+ 65	+ 44	+0.15	+ 16	+ 60	+0.28	+ 22	+ 82
April..........	+0.01	+ 3	+0.23	+110	+113	+0.44	+ 46	+159	+0.53	+ 41	+200
May	+0.30	+101	+0.25	+120	+221	+0.36	+ 38	+259	+0.28	+ 21	+280
June	+0.31	+104	+0.26	+125	+229	+0.21	+ 22	+251	+0.16	+ 12	+263
July	+0.21	+ 70	+0.04	+ 19	+ 89	−0.03	− 3	+ 86	−0.03	− 2	− 84
August	+0.08	+ 26	−0.05	− 24	+ 2	−0.20	− 21	− 19	−0.32	− 25	− 44
September....	+0.02	+ 7	−0.24	−115	−108	−0.26	− 27	−135	−0.40	− 32	−167
October	−0.03	− 10	−0.22	−106	−116	−0.36	− 38	−154	−0.38	− 30	−184
November	−0.15	− 50	−0.25	−120	−170	−0.24	− 25	−195	−0.26	− 21	−216
December	−0.28	− 94	−0.24	−115	−209	−0.04	− 4	−213	−0.05	− 4	−217

TABLE No. 3.—OSCILLATIONS IN MEAN ANNUAL LEVELS OF THE GREAT
PER SECOND FOR EACH PERIOD OF ONE YEAR. THE TABLE
EFFECT AT EACH LAKE BY ADDING

Period.	Lake Superior.		Michigan-Huron.		
	Fluctuation.	Storage.	Fluctuation.	Storage for single lake.	Cumulative storage.
1870-71	—0.36	—10.1	—0.07	— 2.8	—12.9
1871-72	—0.04	— 1.1	—1.28	—51.6	—52.7
1872-73	+0.45	+12.6	+0.65	+26.2	+38.8
1873-74	+0.8	+ 2.2	+0.49	+19.8	+22.3
1874-75	+0.11	+ 3.1	—0.25	—10.1	— 7.0
1875-76	+0.39	+11.0	+1.13	+45.5	+56.5
1876-77	—0.62	—17.4	—0.23	— 9.3	—26.7
1877-78	—0.57	—16	—0.31	—12.5	—28.5
1878-79	—0.44	—13.8	—0.92	—37.1	—50.9
1879-80	+0.33	+ 9.3	+0.14	+ 5.6	+14.9
1880-81	+0.45	+12.6	+0.52	+21.0	+33.6
1881-82	0.00	0.0	+0.37	+14.9	+14.9
1882-83	—0.20	— 5.6	+0.15	+ 6.1	+ 0.3
1883-84	—0.11	— 3.1	+0.10	+ 4.0	+ 0.9
1884-85	+0.25	+ 7.0	+0.25	+10.1	+17.1
1885-86	—0.30	— 8.4	+0.24	+ 9.7	+ 1.3
1886-87	+0.03	+ 0.8	—0.64	—25.8	—25.0
1887-88	+0.40	+11.2	—0.61	—24.6	—13.4
1888-89	—0.14	— 3.9	—0.55	—22.2	—26.1
1889-90	—0.14	— 3.9	—0.11	— 4.4	— 8.3
1891-92	—0.31	— 8.7	—0.56	—22.6	—31.3
1892-93	—0.18	— 5.1	—0.11	— 4.4	— 9.5
1893-94	+0.35	+ 9.8	+0.28	+11.3	+21.1
1894-95	+0.70	+19.7	—0.12	— 4.8	+14.9
1895-96	+0.06	+ 1.7	—1.04	—41.9	—40.2

LAKES, AND THE EQUIVALENT STORAGE IN THOUSANDS OF CUBIC FEET
GIVES THE VALUES FOR EACH LAKE AND THE CUMULATIVE
THE STORAGE IN THOSE ABOVE.

Period.	Erie.			Ontario.		
	Fluctuation.	Storage for single lake.	Cumulative storage.	Fluctuation.	Storage for single lake.	Cumulative storage.
1870–71......	−0.59	− 5.2	−18.1	−1.41	− 9.3	−27.4
1871–72......	−0.96	− 8.5	−61.2	−1.35	− 8.9	−70.1
1872–73......	+0.71	+ 6.3	+45.1	+1.03	+ 6.8	+51.9
1873–74......	+0.51	+ 4.5	+26.5	+0.71	+ 4.7	+31.2
1874–75......	−0.67	− 5.9	−12.9	−1.30	− 8.6	−21.5
1875–76......	+1.42	+12.6	+69.	+1.83	+12.1	+81.2
1976–77......	−0.82	− 7.2	−33.9	−1.17	− 7.7	−41.6
1877–78......	+0.41	+ 3.6	−24.9	+0.58	+ 3.8	−21.1
1878–79......	−0.76	− 6.7	−57.6	−0.31	− 2.0	−59.6
1879–80......	+0.24	+ 2.1	+17.0	−0.39	− 2.6	−14.4
1880–81......	−0.16	− 1.4	+32.2	−0.31	− 2.0	+30.2
1881–82......	+0.87	+ 7.7	+22.6	+1.07	+ 7.0	+33.3
1882–83......	−0.21	− 1.9	− 1.6	+0.11	+ 0.7	− 0.5
1883–84......	+0.07	+ 0.6	+ 1.5	+0.60	+ 4.0	+ 5.5
1884–85......	−0.10	− 0.9	+16.2	−0.45	− 3.0	+13.2
1885–86......	+0.10	+ 0.9	+ 2.2	+0.78	+ 5.1	+ 7.3
1886–87......	−0.03	− 0.3	−25.3	−0.55	− 3.6	−28.9
1887–88......	−0.70	− 6.2	−19.6	−1.25	− 8.2	−27.8
1888–89......	−0.23	− 2.0	−28.1	+0.22	+ 1.5	−26.6
1889–90......	+0.67	+ 5.9	− 2.4	+1.04	+ 6.9	+ 4.5
1890–91......	−0.90	− 8.0	−39.3	−0.95	− 6.3	−45.6
1891–92......	−0.01	− 0.1	− 9.6	−0.74	− 4.9	−14.5
1892–93......	−0.05	− 0.4	+20.7	+0.60	+ 4.0	+24.7
1893 94......	+0.01	+ 0.1	+15.0	−0.20	− 1.3	+13.7
1895–96......	−0.93	− 8.2	−48.4	−1.49	− 9.8	−58.2

TABLE No. 4.—Storage in Thousands of Cubic Feet per Second Represented by a Uniform Rise or Fall of 1 Ft. and of 6 Ins. for Periods Varying from 1 to 6 Months, in the Great Lakes of the St. Lawrence Basin.

Time.	Lake Superior.		Lake Michigan-Huron.				Lake Erie.				Lake Ontario.			
			1 ft.		6 ins.		1 ft.		6 ins.		1 ft.		6 ins.	
	1 ft.	6 ins.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.	Single lake.	Cumulative effect.
1 month....	336	168	482	818	241	409	106	924	53	462	79	1 003	39	501
2 months...	168	84	241	409	121	204	53	462	26	231	39	501	20	250
3 months...	112	56	161	273	80	136	35	308	18	154	26	334	13	167
4 months...	84	42	121	205	60	102	26	231	13	115	20	251	10	125
5 months...	67	34	96	164	48	82	21	185	11	92	16	201	8	100
6 months...	56	28	80	136	40	68	18	154	9	77	13	167	7	84

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PAPERS.

THREE-HINGED MASONRY ARCHES; LONG SPANS ESPECIALLY CONSIDERED.

BY DAVID A. MOLITOR, M. AM. SOC. C. E.

INTRODUCTORY.

The advantages which well-designed masonry arches offer, as compared with the less durable structures of iron and steel, have been adequately demonstrated by modern experience.

The cost of maintenance of iron and steel bridges, together with their more or less limited lasting qualities, are sometimes offset by the ease, simplicity and accuracy of design and erection to which they are susceptible. The time allowable for construction may also, in many cases, weigh strongly in their favor.

However, the many masonry arches built centuries ago—a few antedating written history—are indisputable evidences of permanency. Few of these arches have required any repairs, and their cost of maintenance has amounted to almost nothing, a fact not to be realized in metal bridges.

The purpose of this paper is to demonstrate that masonry arches may be constructed on any good foundation, such as hard clay, and that they will admit of an accuracy and simplicity of design quite

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. The papers with discussion in full will be published in the volumes of *Transactions*.

equal to that attainable for similar structures of iron or steel. In many instances concrete arches are even cheaper in first cost than metal bridges. Besides, the former possess the additional advantages of permanency and low cost of maintenance.

Recent progress, achieved through the earnest labors of German, French and Austrian engineers, has destined the masonry arch to become the successful competitor of iron and steel bridges, whenever the natural conditions of foundations and length of span do not offer unsurmountable difficulties.

The great advances accomplished in the manufacture of cements during the past ten years, the elaborate arch tests made by the Austrian Society of Engineers and Architects from 1890 to 1895, and the construction of a few three-hinged masonry and concrete arches, venturing the adoption of high unit stresses, low factors of safety and long spans; these mark the arrival of a new era in masonry bridge construction.

However, many difficulties are encountered in the construction of fixed masonry arches, owing particularly to insufficient elasticity in the masonry. The natural deformations in the arch, caused by shrinkage of the masonry, setting of mortar, stress and temperature, usually cause cracks which, while rarely of a serious character, are reasons for discouragement to the engineer, who has probably applied every known precaution to prevent their occurrence.

According to the recommendations of the Austrian Society of Engineers and Architects, as a result of their elaborate tests, a fixed masonry arch should be constructed only when the following conditions can be realized:

1. The abutments must be absolutely rigid.

2. The falsework must retain its form during the construction of the arch ring.

3. The material (stone and mortar) must be of the best quality.

4. The construction of the arch ring must be most carefully conducted.

5. The falsework must not be released until the mortar has thoroughly set.

6. When the falsework is released it must be done gradually and uniformly.

These conditions, except the two first named, can always be ful-

filled, though the lack of rigidity of abutments and falsework are the two great obstacles in the way of long-span masonry arches without hinges.

In matters pertaining to the design of fixed masonry arches, it is safe to say that the method based on the theory of elasticity is the only one entitled to full confidence and permitting of an analysis corresponding in accuracy with the knowable properties of the material. All other methods are too approximate to admit of close designing, such as the modern status of engineering science would generally demand.

This modern and most exact method, however, is not free from criticism. While the fundamental principles of the theory are almost axiomatic, their final application to the solution of stresses is extremely complicated, so much so that few engineers can be credited with the patience and earnest endurance to master either the method or the solution of a problem to which it is applied.

Therefore, unless the masonry arch can be so treated as to combine clearness, simplicity, undoubted accuracy and economy in design with faultless construction, the field of usefulness of this class of structure will remain restricted, and such monuments as the Cabin John Bridge will continue to remain curiosities of rare production.

This is not what the masonry arch deserves, in view of its practically everlasting life, nominal cost of maintenance and naturally æsthetic form, which latter should be a prime factor, though rarely given much consideration, in the choice of a bridge.

Essentially all the harassing features of fixed masonry arches are overcome by the introduction of hinges at the crown and abutments, thus permitting a rigid analytical treatment and affording almost absolute safety against cracks, even though small settlements may take place in the abutments. The idea was introduced by Koepke, of Dresden, in 1880, by providing open joints at crown and haunches. Karl v. Leibbrand, Stuttgart, in 1885, substituted sheet lead for the open joints, and in 1893, applied cast-iron, hinged bearings. The author, as early as 1888, while engaged on the construction of the strategical railway Weizen-Immendingen, Baden, Germany, advocated metal hinges for masonry arches, but prejudice and custom prevented a practical application being made at that time.

Some of the noteworthy bridges which have been constructed with hinges or hinge-like joints are briefly described in the following:

1. Bridges on the railroads of Saxony, built in 1880, by Koepke. The largest was of sandstone, 13 m. span, 3 m. rise, thickness of arch ring 0.50 m. to 0.60 m. Hinges consisted of a convex surface of sandstone, radius = 0.977 m., rolling in a concave surface, radius = 1.105 m. The maximum unit stress was 12.87 atm. Several arches of this type were constructed, some with only two hinges and some of concrete. All gave excellent satisfaction.

2. Sandstone bridge over the Enz River near Hoefen, Germany, built by Leibbrand, in 1885. Span, 28 m.; rise, 2.8 m.; maximum stress, 24 atm. Hinge-like joints of sheet lead. Several other bridges of this type were built in 1886 to 1890. The unit stresses were successively increased until 56.4 atm. were attained on the Forbach Bridge, in Baiersbronn, using sandstone of 653 atm. breaking strength.

3. Concrete arch over the Danube River, near Munderkingen, Wurtemberg, built by K. v. Leibbrand, in 1893. Span, 50 m.; rise, 5 m.; thickness of arch, 1 m. at crown, 1.4 m. at quarter points, and 1.1 m. at abutments. This arch was constructed as a three-hinged arch, and was the first masonry arch with actual hinged joints. The maximum compression in the arch was 34.6 atm. and 57 atm. adjacent to the steel hinges. The concrete was composed of 1 part Portland cement to 2½ parts sand and 5 parts broken limestone, showing an ultimate compressive strength of 254 atm. in 28 days and 520 atm. in 2 years and 7 months. The settlement at the crown, from the time of closing the arch ring to the entire completion, was 13.1 cm. One abutment is founded on rock, the other has a pile foundation.

4. Concrete bridge over the Danube near Rechtenstein, Wurtemberg, built in 1893, by Engineer Braun. This bridge is made up of two arches, each of 23 m. span and 2.5 m. rise; the thickness of the arch is 0.65 m. at the crown and 0.9 m. at the haunches. Concrete for arch ring was composed of 1 part Portland cement to 2½ parts sand to 5 parts gravel and ⅛ part quarry stone. The hinges are 18-cm. and 20-cm. lead strips for the crown and abutments, respectively. The highest stress in the arch ring was 18 atm., and the settlements at the crowns of the two spans were 4.0 and 3.0 cm. One abutment was founded on piles, the other on gravel and boulder strata, and the middle pier on solid rock.

5. Bridge de la Coulonvrenière, over the Rhône River, in Geneva, Switzerland, built in 1895, by Engineer Butticaz. This bridge was made of concrete and consists of two main arches spanning 40 m. each, with a rise of 5.55 m. each, separated by a small span of 14 m., and a 12-m. arch adjoining the abutment of one of the main arches. The large spans were patterned after the arch at Munderkingen, and the small spans were supplied with lead joints, as the bridge at Rechtenstein.

6. Concrete bridge over the Danube, near Inzighofen, Wurtemberg, built in 1896, by Max Leibbrand. Span, 43 m.; rise, 4.46 m. hinged

at crown and abutments with cast-iron, hinged pedestals. Thickness of arch ring, 0.7 m. at crown, 1.1 m. at quarter points and 0.78 m. at abutments. Maximum stress, 36.5 atm. in compression, and 1 atm. in tension. Concrete for arch ring was composed of 1 part Portland cement to $2\frac{1}{2}$ parts sand to 4 parts crushed limestone with $\frac{1}{2}$ part limestone screenings. Settlement of arch during construction (4 months) was 8 cm.

All the above-mentioned hinged and semi-hinged arches, besides others which could not be enumerated here, have given excellent satisfaction and have developed no cracks, even though some were founded on piles and others on clay foundations. Age will undoubtenly be beneficial rather than detrimental, which has never been said for iron or steel bridges.

The oldest concrete bridge seems to have been built in Switzerland near Aarau, in 1840, using Roman cement. This bridge has a span of 7.2 m., and a rise of 3 m. Even this cement, which is not as good as most natural cements, has stood the test of time.

With the adoption of three hinges and the evidence just submitted, it will be possible to construct a masonry arch on almost any moderately good foundation and with reasonable assurance against cracks, both during and after construction, all of which should be regarded as a welcome step in advance. This feature also makes it possible to determine the stresses for any system of loading with accuracy and certainty, also to stress the material from one-tenth to one-sixth its ultimate strength as obtained from test samples. All these advantages combined in the three-hinged masonry arch place it on a high plane of engineering perfection. It is hoped that this paper may be the means of introducing this form of arch construction into the United States.

In the following, all dimensions are given in the metric system simply for convenience in computations, but the formulas are equally applicable to any other system of units. The abbreviations used are:

1 meter	= 1 m.	= 3.2809 ft.
1 square meter	= 1 m.^{2}	= 10.7641 sq. ft.
1 cubic meter	= 1 m.^{3}	= 35.3156 cu. ft.
1 centimeter	= 1 cm.	= 0.3937 in.
1 kilogram	= 1 kl.	= 2.2046 lbs. avoir.
1 atmosphere	= 1 atm.	= 0.9482 English atm. = 1 kl. per cm.^{2}:
		= 14.223 lbs. per square inch.

Theoretical Deductions.

I.—General Equations for the Three-Hinged Arch.

(*a*) *Applied Forces and Reactions.*—Given the arch, Fig. 1, with hinged joints at A, B and O, acted upon by the forces P_1 and P_2, assumed to represent the resultants of all vertical loads applied respectively to the left and right of the crown O. Required to find the reactions at the hinges A, B and O.

The thrust produced by P_1 on the segment $O\,B$ must pass through the hinge O, the only point of contact between the segments $A\ O$ and $O\ B$. This thrust must also pass through the hinge B, otherwise the segment $O\ B$ would rotate about B. Also, the intersection c of $O\ B$ with the force P_1 must be a point on the line of the reaction produced

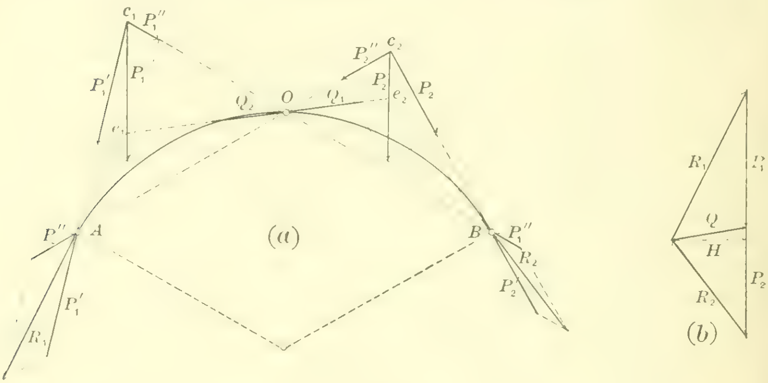


Fig. 1.

by P_1 in A. The reactions in A and B, produced by P_1 are, therefore, P_1' and P_1'', respectively, determined by the resolution of P_1 in the parallelogram indicated.

In like manner the reactions produced by P_2 are found to be P_2'' and P_2', acting in A and B respectively. The resultant reactions in A and B are then R_1 and R_2 respectively, R_1 being the resultant of P_1' and P_2'' and R_2 the resultant of P_1'' and P_2'.

If the forces R_1, R_2, P_1 and P_2 are combined in a force polygon, Fig. 1 *b*, and the equilibrium polygon $A\ e_1\ O\ e_2\ B$ is drawn, it will be seen that the reactions Q are equal and opposite, and that the line of action of $e_1\ O\ e_2$ is a straight line passing through the hinge O. This equilibrium polygon is called the line of thrust for the given forces.

The horizontal components of Q_1, Q_2, R_1 and R_2 are all equal to H, (see Fig. 1 b), which is called the horizontal thrust, and is constant for any point of the arch. The vertical components of R_1 and R_2 are the vertical reactions in A and B respectively, and the vertical component V of Q represents the shear at the crown O. Only these horizontal and vertical components of the reactions in A, O and B will be considered in the following, and will be called "the reactions."

The general expressions for the reactions of a three-hinged arch will now be found.

Assume, in Fig. 2, the segment $O\,B$ removed and a force R, resolved into components H (horizontal) and V (vertical), replacing the action of $O\,B$ on $O\,A$. The moment equation about A is

$$P_1\,d_1 - H f + \frac{V l}{2} = 0 \qquad (1)$$

Similarly assume, in Fig. 3, the segment $O\,A$ removed and the

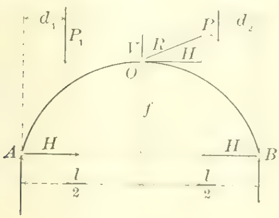


FIG. 2.

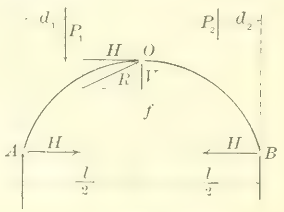


FIG. 3.

equilibrium of segment $O\,B$ preserved by the force R (equal and opposite to R in Fig. 2) resolved into components H and V as before. The moment equation about B is

$$-P_2\,d_2 + H f + \frac{V l}{2} = 0 \qquad (2)$$

evaluating H and V from (1) and (2)

$$H = \frac{1}{2f}\,(P_2\,d_2 + P_1\,d_1) \qquad (3)$$

$$\text{and } V = \frac{1}{l}\,(P_2\,d_2 - P_1\,d_1) \qquad (4)$$

The vertical reactions are obtained as follows from the equations for shear. With reference to Figs. 2 and 3,

$$A = P_1 + V = P_1 + \frac{1}{l}\,(P_2\,d_2 - P_1\,d_1) = \frac{P_1\,(l - d_1) + P_2\,d_2}{l} \qquad (5)$$

$$B = P_2 - V = P_2 - \frac{1}{l}\,(P_2\,d_2 - P_1\,d_1) = \frac{P_2\,(l - d_2) + P_1\,d_1}{l} \qquad (6)$$

From (5) and (6) it will be seen that the reactions A and B are identical with the reactions of a simple beam supported at A and B.

The value of R obtained from either Fig. 2 or Fig. 3 is

$$R = \sqrt{H^2 + V^2} = \sqrt{H^2 + S^2} \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which expression holds good for the resultant thrust at any point of the line of thrust, S being the shear at the point in question.

It follows from (3), that all forces acting on the arch, at points between A and B, affect H positively, while this is not true of the reactions A, B and V.

(b.) Reactions Resulting from a Train of Concentrated Loads, Coming on the Span from the Right-Hand Abutment.—The laws indicated by equations (3), (4), (5) and (6) are applied in deducing the expressions for the general case of loading, Fig. 4, as follows:

$$H = \frac{1}{2f}\left[\Sigma_o^B \; P\,d + \Sigma_o^A \; P\,(l - d)\right] \ldots\ldots\ldots\ldots\ldots (8)$$

$$V = \frac{1}{l}\left[\Sigma_o^B \; P\,d - \Sigma_o^A \; P\,(l - d)\right] \ldots\ldots\ldots\ldots\ldots (9)$$

$$A = \frac{1}{l} \; \Sigma_A^B \; P\,d. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$B = \frac{1}{l} \; \Sigma_A^B \; P\,(l - d) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Also, the shear at any point m, distant a from the crown O is found as for a beam of span $A\,B = l$ and is

$$S = A - \Sigma_m^A \, P = \frac{1}{l} \; \Sigma_A^B \, P\,d - \Sigma_m^A \, P \ldots\ldots\ldots\ldots (12)$$

In the above equations the expression $\Sigma_o^B \; P\,d$ is used to indicate the sum of the products $P\,d$ for all loads acting between the points O and B. Other expressions of summations are to be interpreted accordingly.

(c.) Reactions Resulting from a Uniform Live Load p per Unit of Length, Coming on the Span from the Right-Hand Abutment and Extending over the Span to a Distance e to the Left of the Crown O.—The following equations are obtained analogous to equations (8), (9), (10) and (11):

$$H = \frac{1}{2f}\left[\frac{p\,l^2}{8} + \frac{p\,e\,(l - e)}{2}\right] \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} - \frac{p\,e\,(l - e)}{2}\right] \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

$$A = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

$$B = p\left(\frac{l}{2} + e\right) - A = \frac{p}{2\,l}\left(\frac{3\,l^2}{4} + e\,l - e^2\right) \ldots (16)$$

(*d.*) *Symmetrical Loading.*—For equal loads placed symmetrically with respect to the crown, or for symmetrical loading, equations (3), (4), (5) and (6), also equations (8), (9), (10) and (11), give the following special values:

$$H = \frac{\Sigma_0^A P\,d}{f}, \qquad V = 0 \quad \text{and } A = B = \Sigma_0^A P.$$

For uniformly distributed load over the entire span, $e = \frac{l}{2}$ and equations (13), (14), (15) and (16) give

$$H = \frac{p\,l^2}{8}, \qquad V = 0 \quad \text{and } A = B = \frac{p\,l}{2}.$$

As only symmetrically shaped arches are to be treated in the following, the analysis will be confined to only the half span.

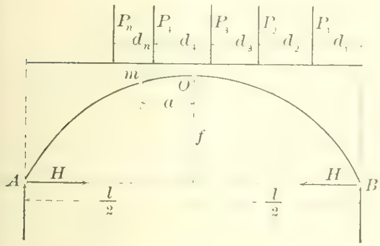


FIG. 4.

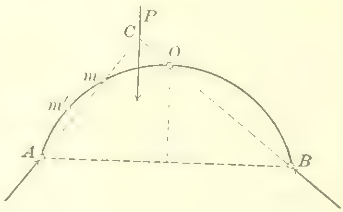


FIG. 5.

II.—Position of a Moving Load for Maximum Stresses.

For any arch of center line $A\ O\ B$ and hinged at A, O and B, Fig. 5, the reaction at B, resulting from any load P to the left of O, will have the direction $C\ O\ B$ and the reaction at A will pass through A and C. The reaction A will then produce a moment about any point m' of the arch center line, which will be negative or positive accordingly as this point m' is above or below the line $A\ C$, and this moment will become zero for a point m on the line $A\ C$. Hence the vertical through C is the dividing line of loads or load divide for positive and negative influences on the moment about m, which point is the intersection of the line $A\ C$ with the center line $A\ O\ B$, though more properly the line of thrust.

Hence, a system of loads covering the span from the right abutment B up to C, will produce maximum compression in the fibers of

the intrados and maximum tension (if any) in the fibers of the extrados for the arch section at m. Also, a system of loads covering the span from the abutment at A up to C will produce maximum compression in the fibers of the extrados and maximum tension (if any) in the fibers of the intrados for this same arch section at m.

Since the resultant thrust in masonry arches is usually confined to the middle third of the arch ring (see Section IV) and accordingly this line of thrust is very nearly normal to the voussoir joints or *radii rectori*, it follows that the shear component of this thrust is necessarily small and any discussion with regard to loading for maximum and minimum shears is considered superfluous, especially as the unit stresses are somewhat liberally chosen, owing to the rather uncertain properties of masonry.

III.—Combined Action of Dead and Live Loads.

(*a*.) *Uniformly Distributed Live Load and Symmetrical Dead Load.*—1. Case of loading for maximum compression in the intrados for any point m of the left half of span: This case of loading will also give the minimum stress in the extrados at m. In accordance with the preceding, the live load must in this case cover the span from the right-hand abutment at D up to the load divide C for the point m, Fig. 6. The following definitions of terms will be adopted and retained throughout this work.

Let m be the point of application of the resultant thrust R of the external forces on any voussoir joint or radial section $m\,n$ under consideration.

Let y and a be the co-ordinates of the point m, referred to the rectangular axes $O\,E$ and $O\,n$ with origin at O.

$\Sigma_o^a\, q$ = resultant dead load of that portion of the span between O and m.

b = distance from O to line of action of $\Sigma_o^a q$.

$\Sigma_o^{\frac{l}{2}}\, q$ = resultant dead load for the half span $O\,A$.

s = distance from A to the line of action of $\Sigma_o^{\frac{l}{2}}\, q$.

p = live load per unit of length, covering the span from D to C.

e = distance from crown O to the load divide C to the left of O.

H = horizontal thrust from total dead load A to B and live load from D to C.

V = shear at O due to live load D to C. The dead load being symmetrical does not affect V.

S = total shear at m from dead and live loads as assumed for H.

R = resultant thrust at m = resultant of H and S. This resultant passes through m and hence its moment about m must be zero, likewise the sum of the moments of the external forces about m must be zero.

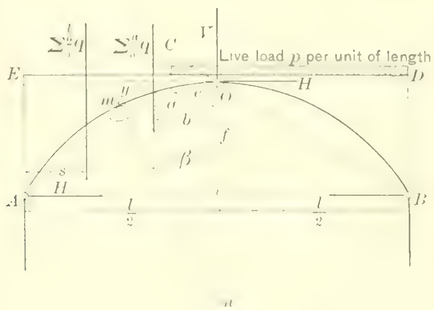


FIG. 6.

Since the dead load is not uniformly distributed in the case of an arch, the quantities, $\Sigma_o^l q$ for successive positions of m are not proportional and must be obtained by summing the q's. The entire arch is divided into vertical segments and the weight q, of each, is determined, as is also the distance, r, from the crown, O, to the center of gravity of each q. The distances, b, are obtained by dividing the sums of the moments, $q\,r$, by the sums of the corresponding q's, thus

$$b = \frac{\Sigma_o^a q\,r}{\Sigma_o^a q}.$$

The moment equation of external forces about m is

$$(a - b)\,\Sigma_o^a q + a\,V - e\,p\left(a - \frac{e}{2}\right) - H\,y = 0$$

which when solved for y gives

$$y = \frac{(a - b)\, \Sigma_0^a\, q + a\, V + e\, p \left(a - \frac{e}{2}\right)}{H}$$

wherein, from equations (13) and (14),

$$V = \frac{1}{l}\left[\frac{p\, l^2}{8} - p\, e \frac{(l - e)}{2}\right] \qquad \text{(17)}$$

and

$$H = \frac{s}{f}\, \Sigma_0^{\frac{l}{2}}\, q + \frac{1}{2f}\left[\frac{p\, l^2}{8} + p\, e \frac{(l - e)}{2}\right]$$

Also, by analogy, with equations (12) and (15),

$$S = A - \Sigma_a^{\frac{l}{2}}\, q = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \Sigma_0^{\frac{l}{2}}\, q - \Sigma_a^{\frac{l}{2}}\, q = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \Sigma_0^a q \qquad \text{(18)}$$

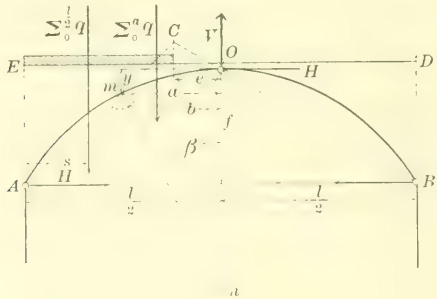


Fig. 7.

Now, since for any point, m, whose absissa, a, is known or assumed, the ordinate, y, can be found from (17), the locus of m, for all points from A to O, is fully determined and can be drawn. This locus represents the extreme positions of all possible lines of thrust resulting from the combined action of dead load and moving live load.

The loci of maximum and minimum effects give at once the data for obtaining the thickness of arch ring for every assumed point, m (see Section IV).

2. Case of loading for maximum compression in the extrados for any point m of the left half of span. The same loading will also give the minimum stress in the intrados at m. In this case the live load must cover the span from the left hand abutment E up to the load divide C, Fig. 7.

Retaining the previous notation, the moment equation for the point m is:

$$(a - b)\, \Sigma_o^a\, q - a\, V + \frac{p}{2}(a - e)^2 - H\, y = 0$$

which gives for y the value

$$\left.\begin{array}{l} y = \dfrac{(a - b)\, \Sigma_o^a\, q - a\, V + \dfrac{p}{2}(a - e)^2}{H} \\ \text{in which} \\ V = \dfrac{p}{2\,l}\left(\dfrac{l}{2} - e\right)^2 \text{ and } H = \dfrac{s}{f}\, \Sigma_o^{\frac{l}{2}}\, q + \dfrac{p}{4\,f}\left(\dfrac{l}{2} - e\right)^2 \end{array}\right\} \ldots\ldots.(19)$$

also

$$\left.\begin{array}{r} S = \Sigma_o^{\frac{l}{2}}\, q + \Sigma_o^a\, q + p\,(a - e) - B = \Sigma_o^a\, q + p\,(a - e) - \\ \dfrac{p}{2\,l}\left(\dfrac{l}{2} - e\right)^2 \end{array}\right\} \ldots.(20)$$

3. Case of loading for the half span covered with uniformly distributed live load for any point m of the left half of span. In this instance the locus of y represents a line of thrust for the assumed case of loading. The condition imposed makes $e = 0$, hence for load extending from D to O, (17) and (18) give

$$y = \frac{(a - b)\, \Sigma_o^a\, q + \dfrac{a\,p\,l}{8}}{H} \ldots\ldots\ldots\ldots\ldots\ldots(21)$$

and

$$S = \Sigma_o^a\, q + \frac{p\,l}{8} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(22)$$

and for load extending from E to O, (19) and (20) give

$$y = \frac{(a - b)\, \Sigma_o^a\, q + \dfrac{p\,a^2}{2} - \dfrac{a\,p\,l}{8}}{H} \ldots\ldots\ldots.(23)$$

and

$$S = \Sigma_o^a\, q + a\,p - \frac{p\,l}{8} \ldots\ldots\ldots\ldots\ldots\ldots(24)$$

In equations (21) and (23) the value of H remains constant for any position of m and has the value

$$H = \frac{s}{f}\, \Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l^2}{16\,f} \ldots\ldots\ldots\ldots\ldots\ldots\ldots(25)$$

This condition of loading was formerly applied as a case for maximum stresses at the quarter points; but as is readily seen from the above, this assumption gives values which are much too small.

4. Case of loading for maximum values of H and S, the entire span being symmetrically covered with uniformly distributed live load. For this condition of loading, the shear V at the crown O becomes zero, and the locus of m represents a line of thrust for the imposed loads.

The equation of moments about any point m on the line of thrust is

$$(a - b)\,\Sigma_o^a\, q + \frac{a^2 p}{2} - H\,y = 0\,;$$

which gives

$$\left.\begin{aligned} y &= \frac{(a - b)\,\Sigma_o^a\, q + \frac{a^2 p}{2}}{H}; \\ \text{in which}\quad H &= \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l^2}{8\,f} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots (26)$$

also

$$S = \Sigma_o^a\, q + a\,p \text{ and } S_{max} = \Sigma_o^{\frac{l}{2}}\, q + \frac{p\,l}{2} = A_{max} \ldots\ldots (27)$$

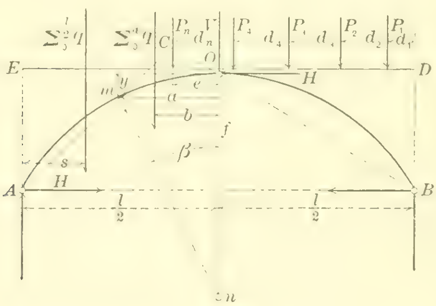


FIG. 8.

(b.) Train of Concentrated Live Loads and Symmetrical Dead Load.

1. Case of loading for maximum compression in the intrados for any point m of the left half of span. Using the notation given under (a) and assuming a system of loads as in Fig. 8, extending over the span from D to the load divide C, the following equations are derived in the manner already indicated for (17) and (18). The same case of loading will also give the minimum stress in the extrados at m.

$$y = \frac{(a - b)\, \Sigma_o^a q + a\, V + \Sigma_o^e P \left(\frac{l}{2} + a - d \right)}{H}$$

in which

$$V = \frac{1}{l} \left[\Sigma_o^{-\frac{l}{2}} P\, d - \Sigma_o^e P\, (l - d) \right] \qquad \ldots\ldots (28)$$

and

$$H = \frac{s}{f}\, \Sigma_o^{\frac{l}{2}} q + \frac{1}{2\, f} \left[\Sigma_o^{\frac{l}{2}} P\, d + \Sigma_o^a P\, (l - d) \right]$$

also

$$S = A - \Sigma_a^{\frac{l}{2}} q = \frac{1}{l} \Sigma_o^{-\frac{l}{2}} P\, d + \Sigma_o^{\frac{l}{2}} q - \Sigma_a^{\frac{l}{2}} q + \Sigma_o^a q + \frac{1}{l} \Sigma_e^{-\frac{l}{2}} P\, d \quad (29)$$

It will be seen from the value of V in (28) that a load P falling exactly at O will neither increase or decrease the value of V.

2. Case of loading for maximum compression in the extrados for any point m of the left half of span. The same loading will also give the minimum stress in the intrados at m. The loads are assumed to cover the span from E to the load divide C, and the distances d are measured as before from the abutment at D. Notation as before, Fig. 8.

$$y = \frac{(a - b)\, \Sigma_o^a q - a\, V + \Sigma_a^e P \left(\frac{l}{2} + a - d \right)}{H}$$

in which

$$V = \frac{1}{l}\, \Sigma_e^{\frac{l}{2}} P\, (l - d) \qquad \ldots\ldots\ (30)$$

and

$$H = \frac{s}{f}\, \Sigma_o^{\frac{l}{2}} q + \frac{1}{2\, f} \Sigma_e^{\frac{l}{2}} P\, (l - d)$$

also

$$S = \Sigma_o^{\frac{l}{2}} q + \Sigma_o^a q - B + \Sigma_e^a P = \Sigma_o^a q + \Sigma_e^a P - \frac{1}{l}\, \Sigma_e^{\frac{l}{2}} (l - d) \quad \ldots\ldots\ldots\ldots (31)$$

In order to obtain the greatest load effects in this and the previous cases, the heaviest loads should be placed near O in case 1, and at C in case 2.

(*c.*) *Loading as Under* (*a*) *Combined with a Concentrated Live Load W.*

1. Case of loading for maximum compression in the intrados for any point m of the left half of span, giving also the minimum stress in the extrados at m. Since a single concentrated load exerts its

maximum positive influence on y when applied just to the right of the crown O (as may be seen from equations (28), the load W will be assumed to act at a unit distance to the right of O for any point m of the left half of span. Hence equations (17) and (18) will apply here when the effect of W is introduced (see Fig. 6).

$$\left.\begin{aligned} y &= \frac{(a-b)\,\Sigma_o^a q + a\,V + e\,p\left(a-\frac{e}{2}\right)}{H} \\ &\text{in which} \\ V &= \frac{1}{l}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2}-1\right) - \frac{e\,p}{2}\,(l-e)\right] \\ &\text{and} \\ H &= \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2\,f}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2}-1\right) + \frac{e\,p}{2}\,(l-e)\right] \end{aligned}\right\} \ldots (32)$$

Also

$$S = \Sigma_o^a q + \frac{p}{2\,l}\left(\frac{l}{2}+e\right)^2 + \frac{W}{l}\left(\frac{l}{2}-1\right) \ldots\ldots\ldots (33)$$

2. Case of loading for maximum compression in the extrados for any point m of the left half of span, producing also the minimum stress in the intrados at m. Here W exerts its maximum influence on V when applied vertically over m, and equations (19) and (20) are modified as follows. See, also, Fig. 7.

$$\left.\begin{aligned} y &= \frac{(a-b)\,\Sigma_o^a q - a\,V + \frac{p}{2}\,(a-e)^2}{H} \\ &\text{in which} \\ V &= \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right] \\ &\text{and} \\ H &= \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2f}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right] \end{aligned}\right\} \ldots\ldots (34)$$

also

$$S = \Sigma_a^o q + p\,(a-e) - \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right] \ldots (35)$$

IV.—Conditions of Stress on a Radial Section of an Arch.

(*a*) *The Resultant Normal Thrust on the Section.*—The resultant thrust R at any point m of a linear arch $A\,O$, Fig. 9, is obtained from equation (7), as $R = \sqrt{H^2 + S^2}$, in which H is the horizontal thrust and S the vertical shear for this point. But as R is not usually normal to the radial section $m\,n$, it will have components perpendicular to and parallel with this section.

The resultant R is resolved into N (the normal thrust) and T (the tangential thrust) respectively perpendicular to and parallel with the radial section $m\ n$, which section is represented by the radius vector of the curve $A\ O$ at the point m and makes the angle β with the vertical.

The values of N and T in terms of H and S are now found without involving R in the result.

The forces S, H, R, N and T are shown in their proper relationship in Fig. 9, from which the following equations are obtained:

the angle $a\ m\ d$ = the angle $a\ b\ d$ = the angle $O\ n\ m = \beta$ also

$$a\ b = S, \text{ and } b\ d = T$$

also

$$a e = c\ d = a\ b \sin \beta = S \sin \beta$$

and

$$m\ c = m\ a \cos \beta = H \cos \beta$$

hence

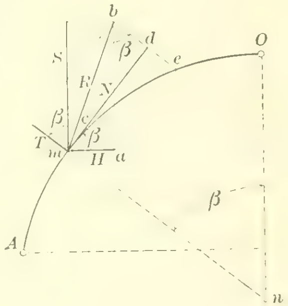


Fig. 9.

$$N = m\ d = m\ c + c\ d = H \cos \beta + S \sin \beta \ldots\ldots\ldots\ldots\ldots (36)$$

and

$$T = b\ d = b\ e - e\ d = S \cos \beta - H \sin \beta \ldots\ldots\ldots\ldots\ldots (37)$$

However, the tangential force T rarely becomes sufficiently large to require any consideration, especially when an arch is so designed that no tensile stresses will ever occur, thereby confining the thrust R to the middle third of the arch-ring and reducing the angle between R and N to a very small quantity. Also the high factors of safety (6 to 10) used in masonry arches would not warrant the consideration of so small a factor as T.

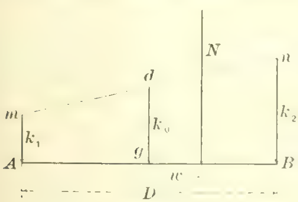


Fig. 10.

(b) *The Stresses on the Section.*—1. Analytical Solution. In Fig. 10, let $A\ B = D$ represent the thickness of the arch on a radial section $m\ n$; also let g be the middle point of $A\ B$, and the point of application of the resultant thrust N, distant w from g. Required the manner in which N is distributed over the section and the intensities of the unit stresses k_1 and k_2 on the extreme elements of the arch-ring.

If N is given for an arch of unit breadth then k_0, k_1 and k_2 will be unit stresses.

For N acting in g, it is evident that the stress is uniformly distributed over $A\,B$ and $k_0 = k_1 = k_2 = \frac{N}{D}$. For N acting at a distance w to the right of g, k_1 will be less than k_0, and k_2 will be greater than k_0, each by an amount $f = \frac{M\,z}{I}$ representing the effect of the moment $N\,w$.

Therefore,

$$k_1 = k_0 - \frac{M\,z}{I} \text{ and } k_2 = k_0 + \frac{M\,z}{I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (38)$$

in which

$$z = \frac{D}{2},\ I = \frac{b\,D^3}{12} = \frac{D^3}{12} \text{ for } b = 1 \text{ and } M = N\,w = w\,D\,k_0.$$

By substituting these values in 38 and reducing

$$k_1 = k_0\left(1 - \frac{6\,w}{D}\right) \text{ and } k_2 = k_0\left(1 + \frac{6\,w}{D}\right) \quad \ldots\ldots\ldots\ldots \quad (39)$$

The lines $m\,d\,n$ represents the manner of distribution of stress produced by the resultant N on the section $A\,B$.

2. Graphical Solution. Equations (39) may be written thus $k_1 = \frac{6\,k_0}{D}\left(\frac{D}{6} - w\right)$ and $k_2 = \frac{6\,k_0}{D}\left(\frac{D}{6} + w\right)$, in which form they may be represented graphically.

In Fig. 11, draw to scale of forces $k_0 = \frac{N}{D}$ at g, perpendicular to $A\,B$, and lay off distances $a\,g$ and $b\,g = \frac{D}{6}$ to the right and left from g. Draw $a\,d$ and prolong same to intersect N in h, also draw $d\,b$ intersecting N in i. Then will $i\,t = A\,m = k_1$, and $t\,h = B\,n = k_2$, and $m\,d\,n$ will represent the law of variation of stress over the section $A\,B$.

$$\text{For } k_1 = k_0 - r = i\,b \tan\alpha = \frac{6\,k_0}{D}\,i\,b = \frac{6\,k_0}{D}\left(\frac{D}{6} - w\right)$$

$$\text{and } k_2 = k_0 + r = a\,t \tan\alpha = \frac{6\,k_0}{D}\,a\,t = \frac{6\,k_0}{D}\left(\frac{D}{6} + w\right).$$

In equations (39) when $w = 0$, $k_1 = k_2 = k_0$, and when $w = \frac{D}{6}$, $k_1 = 0$ and $k_2 = 2\,k_0$. When $w > \frac{D}{6}$, k_1 becomes negative or tensile. Hence to avoid tensile stresses on any section $A\,B$, the resultant normal thrust N must have its point of application t within the middle third $a\,b$ of said section.

(c) *Thickness of Arch Ring.*—Given the direction, amount and point of application of the normal thrusts N_e and N_i, obtained from the loading for maximum compression in the extrados and intrados respectively, for a radial section $A\ B$, Fig. 12, to find the thickness D of the arch ring which must be provided so that a certain assigned unit stress k on the extreme elements of the ring shall never be exceeded.

While N_e and N_i can never occur simultaneously, the center line of the arch must be so placed, with respect to these thrusts, and a minimum value of D, that the above conditions may be complied with.

The dimension e in Fig. 12 is obtained from the difference between maximum and minimum y, which vertical difference is projected on the radial section $A\ B$ by multiplying with $\cos\ \phi$; hence $e = \Delta\ y \cos\ \phi$ is a known quantity, and w and w' are to be found, likewise D.

The values k_o in equations (39) for the two thrusts are

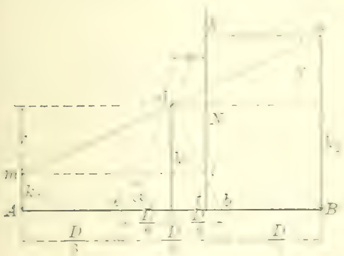


FIG. 11

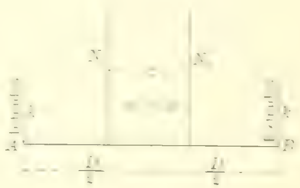


FIG. 12.

$k_o = \frac{N_e}{D}$ and $k_o' = \frac{N_i}{D}$, and from the figure $w' = e - w$. These values substituted in the second equation (39) give the two following values of $k_2 = k$ the given allowable compressive stress:

$$k = \frac{N_e}{D}\left(1 + \frac{6\ w}{D}\right) \text{ and } k = \frac{N_i}{D}\left(1 + \frac{6\ (e - w)}{D}\right)$$

These equations when solved for D and w give the dimensions sought.

$$D = \frac{N_e}{k\left(1 + \frac{N_e}{N_i}\right)} + \sqrt{\frac{6}{k}\left(\frac{N_e\ e}{1 + \frac{N_e}{N_i}}\right) + \left[\frac{N_e}{k\left(1 + \frac{N_e}{N_i}\right)}\right]^2} \quad \ldots\ (40)$$

$$w = \frac{k\ D^2}{6\ N_e} - \frac{D}{6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (41)$$

The ordinate of the arch centerline for the section $A\ B$ is also found from w, and the ordinate to the point of application of N_e which

may be called y_{min}. Calling the ordinate of the center line y_c, and the inclination of $A\ B = \beta$ from the vertical,

$$y_c = \frac{u}{\cos\ \beta} + y_{min} \quad \text{.................} \quad (42)$$

The values of k_1 for the above thrusts and dimensions may now be found from the first of equations (39).

(*d*) *Tensile Stresses.*—It may occur that after having found D from (40) and k_1 from (39), the latter value may be negative or tensile. Should this tensile stress be in excess of the allowable stress then the thickness D must be further increased, or if the material of the arch be concrete the excessive tension may be taken up by the insertion of wire netting or iron rods. The method of computing the area of metal required is given in the following:

In Fig. 13, let $A\ B$ be the radial arch section k_1 a tensile stress on the extreme element at A and k_2, the corresponding compressive stress at B.

$a\,b$ is a steel wire placed at a distance r from the axis of the arch. Let a be the area of the steel required for an arch ring of unit breadth and $f =$ the allowable unit stress for steel.

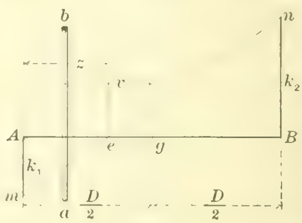


FIG. 13.

The total tensile stress on the arch ring (for unit breadth) will be represented by the area $A\ m\ e = \frac{k_1\,z}{2} = u'$.

From the similar triangles $A\ m\ e$ and $B\ e\ n$

$$\frac{z}{D - z} = \frac{k_1}{k_2}, \text{ or } z = \frac{k_1\ D}{k_1 + k_2}; \text{ making } u' = \frac{k_1^2\ D}{2\ (k_1 + k_2)} \quad \text{.......} \quad (43)$$

The point of application of u' is the center of gravity of the triangle $A\ m\ e$ which is distant from g by an amount $\frac{D}{2} - \frac{z}{3}$. Hence the moment of u' about g is $M = u' \left(\frac{D}{2} - \frac{z}{3}\right)$ which, when taken up by the steel rod, produces therein the force

$$u = \frac{u' \left(\frac{D}{2} - \frac{z}{3}\right)}{r} \quad \text{..................................} \quad (44)$$

But

$a = \frac{u}{f}$; hence from (43) and (44) the value of a is found as

$$a = \frac{k_1^2 D \left(\frac{D}{2} - \frac{k_1 D}{3 (k_1 + k_2)} \right)}{2 f r (k_1 + k_2)} \quad \text{....... ....(45)}$$

In equation (45) k_1 is tensile stress and k_2 is compressive stress, and both enter into the equation without regard to sign of stress.

V.—Deformations of the Arch Ring.

(*a*) *General Consideration; Changes in Length of the Arch Ring.*—An arch ring, when under the influence of stress and changes in temperature, is subject to elastic deformations. The compressibility of masonry by stress and the shrinkage caused by the setting process of mortar and concrete, bring about a permanent shortening in the arch ring during construction and test loading.

The resultant effect of these several shortenings and temperature changes produces a deformation of the arch ring which must be provided for in the construction, to prevent a deflection below the normal, which, if it occurred, would materially increase the horizontal thrust on the abutments. This superelevation, which must be given to the arch ring in order that it may attain its proper rise when completed, is called camber.

Besides the above deformation, which is partly elastic and partly permanent, provision must also be made for the settlement in the falsework caused by the weight of the arch ring up to the time of closing, when the latter becomes self-sustaining. This part of the problem is here omitted as depending entirely on the nature of the falsework and local building conditions.

The change in length of the arch ring resulting from stress, temperature, compressibility of material and shrinkage of masonry will now be found.

The normal arch thrust N may be found, for any combination of dead and live loads, from the preceding equations, and this thrust increases from the crown toward the haunches. The cross-section of the arch ring is also a variable quantity. Hence it is necessary to divide the arch ring into sections over which the cross-section and normal thrust may be considered constant. The sum of the increments of change of these sections gives the change in length of the arch ring.

Let L = length of a section of arch, over which the area and thrust are assumed constant.

$\triangle L$ = increase in the length L for any assigned reasons. Decrease is negative.

$\delta = \Sigma_0^{\frac{l}{2}} \triangle L$ = sum of the changes $\triangle L$ from the crown to the abutment.

N = normal thrust acting through the length L.

F = average cross-section of the section of length L.

E = modulus of elasticity of the material for the working stress $\frac{N}{F}$.

E' = modulus of permanent set of the material for the working stress $\frac{N}{F}$.

t = a rise, and $-t$ = a fall, in temperature, in degrees Centigrade.

α = coefficient of expansion for 1° Centigrade.

e = coefficient of shrinkage from setting of mortar or concrete in air.

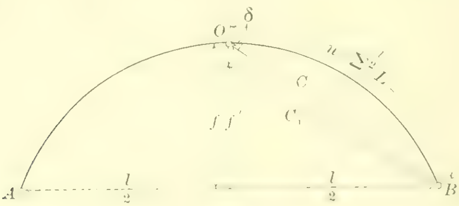


Fig. 14.

then

$$\triangle L = -\frac{N L}{E F} - \frac{N L}{E^1 F} - e L + \alpha t L$$

$$= -\frac{N L}{F}\left(\frac{1}{E} + \frac{1}{E^1}\right) - L\left(e - \alpha t\right) \text{ .... } (46)$$

and $$\delta = -\left(\frac{1}{E} + \frac{1}{E^1}\right) \Sigma_0^{\frac{l}{2}} \frac{N L}{F} - \left(e - \alpha t\right) \Sigma_0^{\frac{l}{2}} L \text{ ...... } (47)$$

The elastic change in length of an arch ring in a completed structure no longer subject to increased permanent set is

$$\delta' = -\frac{1}{E} \Sigma_0^{\frac{l}{2}} \frac{N L}{F} + \alpha t \Sigma_0^{\frac{l}{2}} L \text{ .... } (48)$$

b. Deformation; Analytical Solution.—To find the deflection at the crown of an arch ring, which latter has shortened an amount δ, as found from (47) or (48).

Let C, Fig. 14, be the chord from crown to abutment, correspond-

ing to the half span $\frac{l}{2}$ and rise f; $C_1 =$ the length of this chord after the half arch ring has shortened an amount δ and the rise has diminished to f^1, the span remaining unchanged; also, $n = \Sigma_0^{\frac{l}{2}} L =$ the length of the center line of the arch ring between the crown and the abutment.

A shortening δ in the length n will produce a shortening $\frac{C}{n}\,\delta$ in the chord C, making $C_1 = C - \frac{C}{n}\,\delta$.

From the figure $C = \sqrt{f^2 + \left(\frac{l}{2}\right)^2}$, which, substituted in the previous equation, gives $C_1 = \left(1 - \frac{\delta}{n}\right)\sqrt{f^2 + \left(\frac{l}{2}\right)^2}$ . . . . . . (49)

and

$$\Delta f = f - f^1 = f - \sqrt{C_1^2 - \left(\frac{l}{2}\right)^2} \quad \ldots\ldots (50)$$

The value Δf, which represents the deflection at the crown, may then be found from equations (49) and (50).

While the value $\frac{C}{n}\,\delta$ is not exactly correct, yet the approximation is so close that the resulting equations are entirely within the knowable accuracy even for long spans.

As will be noticed, this solution applies to arches of any shape not necessarily circular, but is most accurate for circular arches.

(c) *Deformation; Graphical Solution.*—The general case for any condition of unsymmetrical loading and displacements of abutments can best be solved by the graphical method.* Only the application of the method is here given, without repetition of its derivation, for which see above-named article.

In Fig. 15*a*, let $A\ O\ B$ represent the line of thrust of a three-hinged arch for any particular case of loading; L_1, L_2, L_3, etc., the sections into which the arch is divided; and $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., the contractions in the lengths L_1, L_2, L_3, etc., respectively, caused from any combination of conditions, as stress, shrinkage of masonry, temperature, etc.

* See article "Distortion of a Framed Structure," by David Molitor, in Journal of the Association of Engineering Societies, Vol. xiii, p. 310.

In Fig. 15 *b*, draw in succession, from any point A', the contractions $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., respectively, parallel to the elements L_1, L_2, L_3, etc., of Fig. 15 *a*, and in the direction in which these contractions act relatively to the fixed point A. In the example, the $\triangle$'s being negative, the elements of the arch, Fig. 15 *a*, all move toward A, and hence the quantities $-\triangle 1$, $-\triangle 2$, $-\triangle 3$, etc., are applied downward from A'. The broken line $A'\ O'$ then represents the motion of the point O relatively to the point A, assuming that the arch elements move parallel to themselves. In like manner, the broken line $O'\ B'$, Fig. 15 *c*, represent the motion of the point O relatively to the point B.

However, the elements of the arch do not move parallel to themselves, but the half arch $A\ O$ revolves about A and the half arch $O\ B$ revolves about B until the point O, common to both halves, attains its new position. This revolution is performed by a second operation as follows:

In Fig. 15 *b*, draw $A'\ O''$ perpendicular to $A\ O$ and through A'; also through O' draw $O'\ r$ perpendicular to $A'\ O''$, and in Fig. 15*c*, through B' draw $B'\ O''$ perpendicular to $B\ O$, and through O' draw $O'\ s$ perpendicular to $B'\ O''$. Now transfer $O'\ s$ from Fig. 15 *c* to Fig. 15 *b*, parallel and equal to itself, and in Fig. 15 *b* draw $s\ O''$ perpendicular to $O'\ s$, and $O''\ O'$ will represent the direction and amount of the displacement of the point O. Also transfer $O'\ r$ from Fig. 15 *b* to Fig. 15*c*, and in Fig. 15 *c* draw $r\ O''$ perpendicular to $O'\ r$, and $O''\ O'$ will check in direction and amount the value $O''\ O'$ found in Fig. 15 *b*.

To find the displacements of the individual points 1, 2, 3, etc., draw a broken line A', $1''$, $2''$, $3''$, $4''$, O'' with its elements proportional and respectively perpendicular to the elements A, 1, 2, 3, 4, O. The lines $1''\ 1'$, $2''\ 2'$, etc., will represent the true displacements of the the points 1, 2, etc., respectively. The same construction applied to Fig. 15 *c* gives the displacements of the points of the half arch $O\ B$.

When the abutments are displaced by amounts $-\triangle A$ and $-\triangle B$. respectively, these displacements are embodied in the diagrams of Figs. 15, *b* and *c*, as indicated by the dotted construction, and the displacements are then measured from the dotted broken lines O_1'', A_1', and O_1'', B'_1, respectively, to the broken lines $O'\ A'$ and $O'\ B'$.

For symmetrical loading, where the displacements in the half arch $A\ O$ are exactly equal to those in the half arch $B\ O$, the line of thrust is symmetrical, and the deflection at the crown O must be vertical.

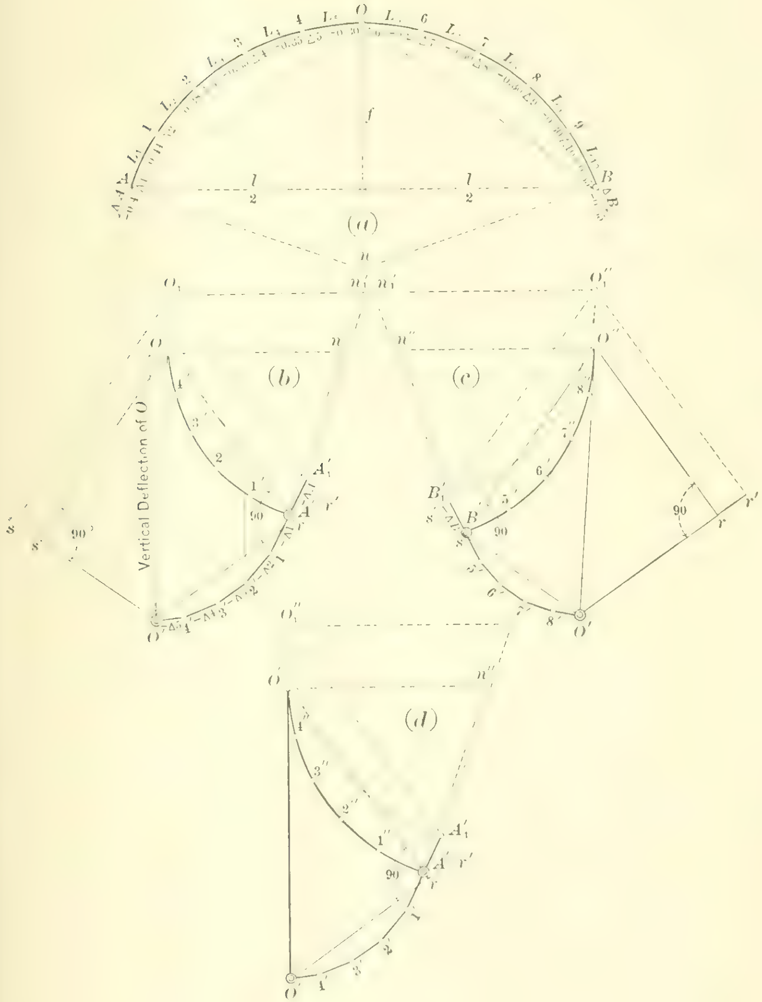


FIG. 15.

This extra condition makes it possible to dispense with one of the diagrams, and the solution becomes as shown in Fig. 15 *d*. The point O'' is then the intersection of $A' O''$ perpendicular to $A O$, and the vertical $O' O''$ through O' becomes the deflection at the crown.

It should be noticed in the solution Figs. 15, that Fig. 15 *a* is drawn to a small scale, while the displacement diagrams *b*, *c* and *d* are drawn to natural scale. For small contractions these diagrams may be drawn to a scale of 10 to 100 times the natural.

(*d*) *Values of e, α, E and E' in Equations (46) to (48).* While it is always best to make accurate determinations of the above values for the material to be used in any particular structure, especially when close agreement is desired, yet a brief summary of the meagre data on this subject may be useful as furnishing values for preliminary computations.*

Values of e for German Portland cement mortars, sixteen weeks old.†

Mortar mixed 1 part cement to 0 part sand..$e = 0.0012$ to 0.0034
" 1 " 3 " ..$e = 0.0008$ " 0.0015
" 1 " 5 " ..$e = 0.0008$ " 0.0014

Values of α for one degree of temperature (Centigrade).

Cement mortar (Bruniceau).............. $\alpha = 0.00001$ to 0.000014
Portland cement concrete 1: $2\frac{1}{2}$: 5 (Bauschinger)............................. $\alpha = 0.0000088$
Stone and brick (Bruniceau) .... $\alpha = 0.0000053$ to 0.0000083

Regarding the values of E and E' it will be necessary to recite briefly the results of the interesting set of experiments made in 1894 by Professor C. Bach, of Stuttgart, which are of vital importance to the subject here treated.

These experiments showed that any concrete when subjected to stress would undergo a permanent set and an elastic deformation, the magnitudes of which become constant after several repetitions of the same stress. The number of repetitions necessary to produce constant deformations appeared to be a function of the breaking strength of the concrete, and of the intensity of the applied stress. The greater the ultimate strength, the less repetitions were required; and the greater the applied stress, the greater the number of repetitions.

* See article by David Molitor on "Properties of Concrete under Compressive Stress," Jour. Assoc. Eng. Soc., 1898.

† See report of Committee on Compressive Strength of Cements, etc. *Transactions* Am. Soc. C. E., Vol. xv, pp. 717.

However, for each kind of concrete a maximum stress was reached (about 0.7 of breaking stress) for which seven to eight repetitions would still continue to increase the deformations. Presumably more repetitions would have restored constancy, but this point may be regarded as a natural limit of allowable stress, though there does not appear to be a definite limit of elasticity.

The tests were made on cylindrical samples of concrete, 1 m. long and 25 cm. in diameter, mixed 1 part Portland cement to 2½ parts sand to 5 parts broken limestone, and 1 part cement to 3 sand to 6 stone, age two to three months. The breaking strength was in every case determined from the cylindrical samples. The strength of the same concrete developed by cubic samples would have been about one and one-half times the strength obtained from the long cylinders.

The following tables give values of E and E' for various values of ultimate strength and applied loads. These values are given in metric atmospheres.

VALUES OF E IN THOUSANDS (Three ciphers should be added to tabulated values).

Ultimate Strength in Atmospheres.		Applied Loads in Atmospheres, or Kls. per Sq. Cm.							
Found from cylinders.	Estimated for cubes.	8	10	15	20	25	30	35	40
60	90	228	223	214	204	193	181	171	163
80	120	262	258	246	234	222	210	200	191
100	150	296	290	278	266	255	244	235	224
120	180	320	314	298	286	276	267	257	247
140	210	340	333	317	303	291	280	270	260

VALUES OF E' IN THOUSANDS (Three ciphers should be added to tabulated values).

Ultimate Strength in Atmospheres.		Applied Loads in Atmospheres, or Kls. per Sq. Cm.							
Found from cylinders.	Estimated for cubes.	8	10	15	20	25	30	35	40
60	90	1 230	1 180	1 030	910	810	720	630	545
80	120	1 800	1 710	1 500	1 320	1 180	1 020	870	755
100	150	2 500	2 400	2 100	1 880	1 680	1 460	1 300	1 160
120	180	3 400	3 210	2 800	2 500	2 220	2 010	1 820	1 650
140	210	4 400	4 190	3 600	3 160	2 780	2 500	2 280	2 100

The important conclusions arrived at by Tourtay,* regarding compressive properties of masonry, are here given.

* Ann. des Ponts et Chaussées, 1885, ii, p. 15.

1. The ultimate strength of small cubes of cement mortar is considerably less than the compressive strength of blocks of masonry made with the same mortar.

2. The pressure which crushes the masonry is an inverse function of the thickness of the mortar joint.

3. Stone plates laid loosely upon each other have a much smaller compressive strength than solid cubes.

4. The same stone plates, when cemented together with neat cement grout, possess the same compressive strength as do solid stone samples.

Practical Applications.

Solution of a Problem.

(*a*) *Introductory.*—To illustrate more clearly the method of arch construction as proposed in the foregoing, a problem is solved in sufficient detail to bring out the practical applications of the theory.

In designing metal bridges, the dead load is generally known, the more accurate for the forms most in use. This is not the case with masonry arches for reasons of insufficient experimental data. Also, a steel arch may be designed with any rational center line because tensile stresses can be provided for, while in masonry only compressive stresses are safely permissible; therefore, the shape of a masonry arch is of necessity a function of its loading.

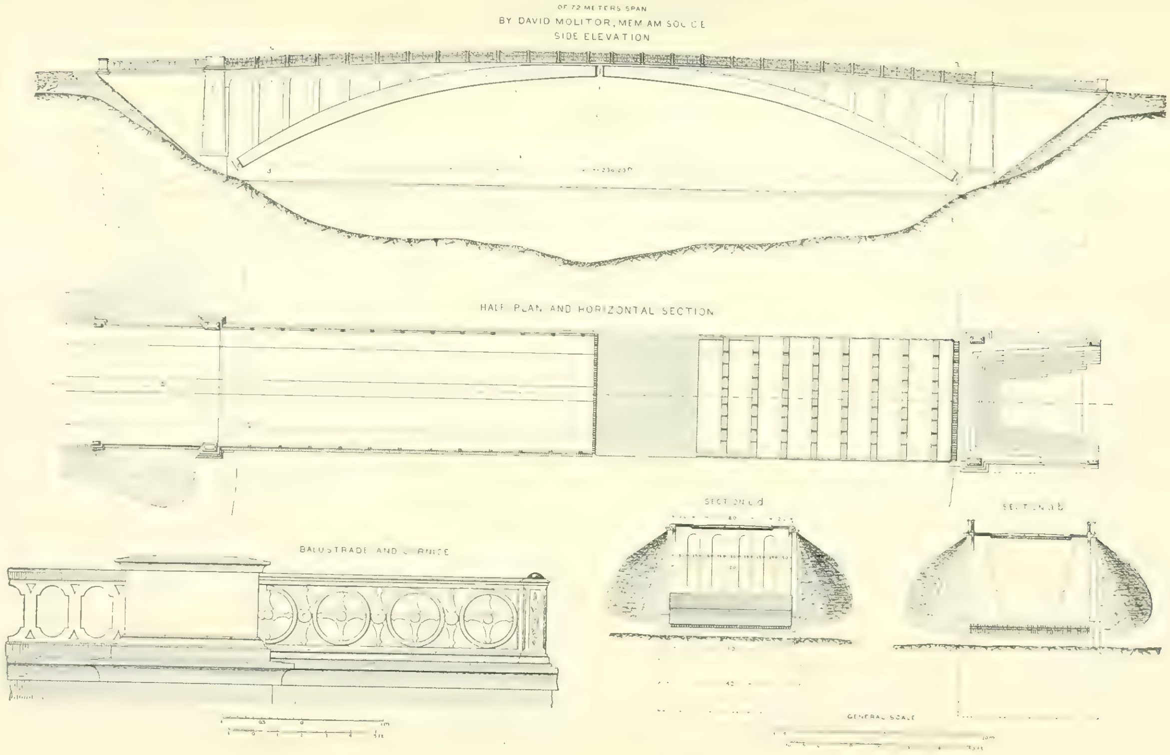
However, with the dead load known, the three-hinged masonry arch can be analysed and dimensioned with the same degree of accuracy, consistent with the nature of the material, as is possible in steel. But, as the dead load can be obtained in no other way than by computation from assumed dimensions, the solution must be reached by successive approximations of dimensions until the loading resulting therefrom produces stresses in the arch which are not in excess of the allowable unit stresses. Much depends on the experience of the designer as to the rapidity and directness with which a solution may be obtained.

The method which it is believed will lead to a solution with a a minimum of useless computation is illustrated in the following problem.

(*b*) *Statement of the Problem.*—Design a three-hinged concrete arch of 72-m. span between pin centers, rise, 10 m., width of driveway,

PLATE XLV.
PAPERS AM. SOC. C. E.
JUNE, 1898.
MOLITOR ON THREE-HINGED MASONRY ARCHES.

DESIGN FOR A THREE HINGED CONCRETE ARCH.



8 m., and footwalks 2 m. wide, on each side of the driveway. The profile of the site is given, and the abutments are to be founded on hard clay. The bridge is to carry an electric motor car weighing 20 000 kl. (44 000 lbs.) on a track of 1.43 m. (4-ft. 8½-in.) gauge, and a uniformly distributed live load of 400 kl. per m.^{2} (82 lbs. per square foot).

The maximum allowable working stresses are 40 atm.* (568.8 lbs. per square inch) in compression, and 2 atm. (28.4 lbs. per square inch) in tension, for concrete composed of 1 part Portland cement to 2 parts sand to 3 parts crushed limestone. This concrete must attain the following compressive strengths on 20-cm. cubes: 220 atm. in 28 days, 350 atm. in 6 months, and 500 atm. in 2 years. The tensile strength of mortar composed of 1 part cement to 3 parts sand must be at least 20 atm. in 28 days.

(c) *Outline of the Method.* (See Plates XLV and XLVI.)—In this as in all bridge designs, the details of the roadway and its supports on the arch ring are first designed. By so doing the dead loads may be computed, involving the weight of the arch ring as the only variable subject to correction.

A diagram of the half span (see dimension diagram Plate XLVI) is then prepared and the roadway and its supporting columns drawn, to which is fitted an arch ring of seeming good proportions for the given span and rise. Herein, the experience of the designer is practically his only guide, but reference to a completed design or a solution once made will assist wonderfully in making close approximations. It will be seen from the side elevation, Plate XLV, that the center line (a three-center curve) is almost a complete circular arc, slightly flattened at the crown.

The concentrated dead loads, q, are now computed by using the dimensions scaled from the preliminary diagram. These values, together with values of a, d and e, are tabulated together with computed values of $q\,d$ (see Table No. 1).

Then assuming the maximum case of loading over the entire span, compute S_{max} from equation (27), H_{max} from equation (26) and $N_{max} = H_{max} \cos \beta + S_{max} \sin \beta$, from which may be found the required thickness of arch ring at the crown $D = \frac{H_{max}}{k}$ and at the abutments

* Stresses and pressures will be given throughout this problem in metric atmospheres equal to 1 kl. per cm.^{2} (14.22 lbs. per square inch).

$D_A = D_B = \frac{N_{max}}{k}$. If it appears that the assumed dimensions differ widely from those computed, it is well to make a second approximation before proceeding further with the solution, though no correct idea can yet be formed of the required arch thickness at the quarter points. The arch thicknesses, at the crown and springing, should be chosen about 10% greater than found by computation, since these sections will be adjacent to the hinges and some allowance must be made for unequal distribution of pressure due to friction in the hinged bearings.

When the solution appears satisfactory to this stage, the assumed arch ring should be tested at two or three points between the crown and springing. The maximum thickness of the arch ring will occur at a distance of about 0.3 l from the crown. This and two other sections, distant about 0.15 l and 0.4 l from the crown, should be dimensioned, after which it will usually be possible to proceed to the final design and computation, unless the agreement with the preliminary design was not sufficiently close, in which case the process should be repeated by approximating a new arch ring, using the dimensions last obtained. In this latter case some allowance should be made for the probable effect of the new dead loads. The larger the ratio between dead and live loads, the less will be the divergence $\triangle y$ of maximum and minimum thrusts, though an increase in dead loads will also increase the resultant normal thrust N_r on any section.

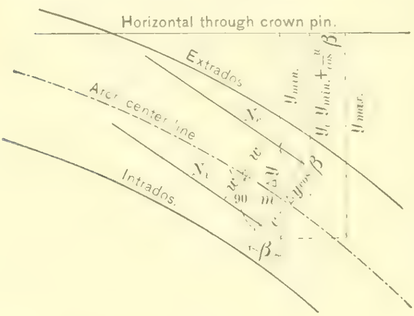


Fig. 16.

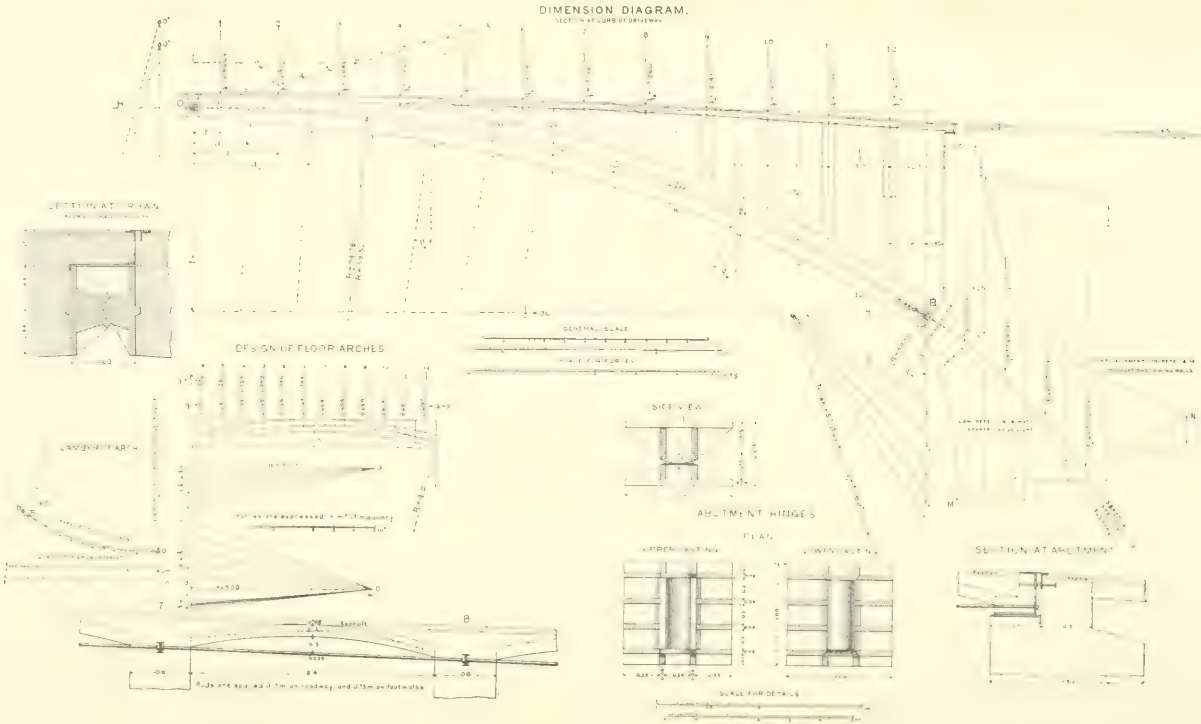
The method of dimensioning just referred to consists in computing maximum and minimum y's and the corresponding H's for each of the three points, using in the present problem equations (32) and (34). The differences of the y's give the $\triangle y$'s in Fig. 16. Then, finding S from equations (33) and (35) for each of the three points, compute N_r and N_r from equation (36). From equations (40), (41) and (42) the values D, w and y_c, respectively, may be found. The value of c in equation (40) is equal to $\triangle y \cos \beta$, as may also be seen in Fig. 16.

PLATE XLVI.
PAPERS AM. SOC. C. E.
JUNE, 1898.
MOLITOR ON THREE-HINGED MASONRY ARCHES.

DESIGN FOR A THREE-HINGED CONCRETE ARCH.

OF 72 METERS SPAN

BY DAVID MOLITOR, MEM AM SOC C E.



All the factors for five points, including crown and springing, being now fully determined, the intermediate values can easily be interpolated in making the final dimension diagram.

The new diagram is now accurately drawn, and the design carefully computed, in the manner just outlined for the three test points, by application of the method to as many points as may be deemed necessary or desirable to fully determine the dimensions and stresses throughout the arch.

There is still one other point which is interesting to know and is illustrated in the final computation. The center line of the arch ring and the line of thrust for the case of one-half the uniformly distributed live load or $\frac{p}{2}$ acting over the entire span, are so nearly alike that no appreciable error would be introduced by accepting this line of thrust for the center line.

The above outline of the method to be followed is now applied to the problem in hand. In the following computations, the arch ring is assumed 1 m. wide, and the live load thus becomes 400 kls. per meter, combined with a concentrated live load of $W = 10\ 000$ kls.

(*d.*) *Design of the Roadway.*—From æsthetic considerations and for economic reasons, the roadway is designed to the parabolic curve whose equation is $-y = 0.001\ x^2$, making the same 1.3 *m.* higher at the crown than at the abutments (see Plate XLV).

The roadway is made up of concrete floor arches of 2.4 m. span, carried by small concrete piers resting on the arch ring. The cross-section of the driveway is the arc of a circle having a middle ordinate of 0.12 m., and is covered with 6 cm. of asphalt composition. The footwalks are sloped toward the bridge axis with a slope of 1 : 100, and are finished in cement mortar. The car track is placed in the center of the driveway and hard paving brick laid adjacent to the rails.

The horizontal thrust of the floor arches is taken up by steel rods 2.5 cm. in diameter, and spaced 17 cm. apart under the driveway and 75 cm. under the footwalks. Expansion joints are provided in the roadway at the crown and at the abutments, all as shown on Plate XLVI. Drainage of the roadway is provided by cast-iron pipes placed in the first piers adjacent to the abutments, one for each gutter.

In accordance with this design the weight of the floor, complete, to the springing of the arches and between the pier centers, is estimated as follows:

Concrete for arch and portion of pier to springing, 0.57 m.^{3} at 2 300 kls.	1 311 kls.
Six steel rods, 2.5 cm. diam., 3 m. long	71 "
Rails and I-beam	105 "
Six cm. asphalt composition at 2 200 kls. per m.^{3}	396 "
Total weight to be carried by one pier	1 883 kls.

The round number 1 900 kls. was used.

The floor arches are designed to carry 10 000 kls. evenly distributed over the half span (unsymmetrical load $q + p$, diagram Plate XLVI), and the center line corresponds to the line of thrust following from 10 000 kls. distributed over the entire span (loads $q + \frac{p}{2}$). These lines of thrust for these loads are drawn for arch horizontal, also for the inclined position between piers 7 and 8. The maximum horizontal thrust is 9 m.^{3} of concrete or 20 700 kls. for an arch 1 m. wide. The maximum compression at the crown is 17 atm. and at the haunches and quarter points it is 36.8 atm. No tension occurs anywhere.

The longitudinal section investigated is at the curb of the driveway where the minimum thickness of arch (15 cm.) is possible. However, this portion of the roadway, according to the above computation, is amply strong to carry a 20-ton road roller. As a result of the crowning of the roadway the floor arches attain a thickness of 27 cm. in the axis of the bridge.

(*e*) *Design of the Arch Ring.*—A preliminary diagram similar to that shown on Plate XLVI, is now drawn and the dead loads q are computed and tabulated, together with values of a, e, r and β, as far as these may be necessary for the points 3, 6 and 10 in the present problem.

The points 4, 7 and 10 would have been somewhat better, but those chosen are quite suitable for good interpolations of intermediate values.

The preliminary investigation for the crown, the points 3, 6 and 10, and the springing, is given in Table No. 1, the results of which indicate fully to what extent the assumed design was in error.

In the preliminary diagram certain values of D and y_o were assumed. These are tabulated together with the results just found.

Point.	Preliminary Values.		Computed Values.	
	D	y_c	D	y_c
	m.	m.	m.	m.
0	0.82	0	0.880	0
3	1.40	0.38	1.436	0.423
6	1.63	1.90	1.660	1.964
10	1.40	6.30	1.500	6.394
12	1.00	10.00	1.060	10.000

From this comparison, which shows a remarkably close first approximation of shape and thickness of arch ring, it will be seen that it is perfectly now safe to proceed to the final design and the detailed computation thereof. This could have been done even with a less satisfactory coincidence in the above values of D and y_c.

From the values of D and y_c just found in Table No. 1, the intermediate values must now be interpolated, preparatory to constructing the final dimension diagram on Plate XLVI.

To find the arch center line, plot the five points, whose co-ordinates a and y_c are now known, and connect them by circular arcs using the method indicated in Fig. 17. Connect the plotted points by straight lines forming the polygon O — 3 — 6 — 10 — 12. Then inscribe in any circle with convenient center $\times$ on the vertical, through O, a polygon O' — 3′ — 6′ — 10′ — 12′, having its sides respectively parallel to those of the foregoing polygon. The radii $\times$ — 3′, $\times$ — 6′, etc., will be parallel to the required radii $\times_3$ — 3, $\times_6$ — 6, etc., and the latter, when drawn, will intersect in the centers $\times_3$, $\times_6$, $\times_{10}$ and $\times_{12}$ of the arcs sought. Usually a three center curve will be found to fit an arch of this type, as was done in the present example (see Plate XLVI).

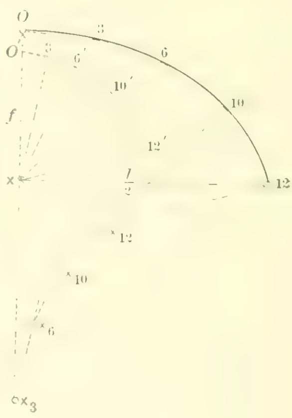


Fig. 17.

To interpolate values of D between those already found, plot the found values as ordinates with the corresponding values of a as in Fig. 18, choosing a vertical scale about twenty times the horizontal. By joining these points with the use of an irregular curved ruler, and noting that maximum D should occur at $0.3\ l$ from the crown, or at

about point 7, the intermediate values of D may then be scaled from the diagram with considerable accuracy.

The diagram, Plate XLVI, can now be drawn, and all necessary dimensions for the final computation are then scaled therefrom. Any slight differences that may be found between these scaled dimensions and the final computed arch dimensions will be too small to warrant a reconsideration, especially as the knowable accuracy with which the dead loads can be determined is far in excess of any differences still to be expected at this stage of the solution. The final drawings should, of course, be constructed with the use of the dimensions resulting from the final computation.

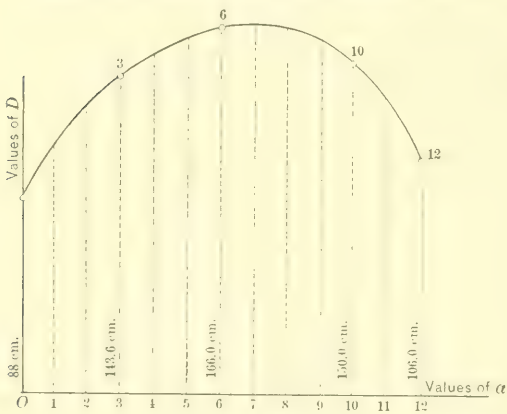


Fig. 18.

The final computation is carried out in complete detail in the following tables, and all the steps in the solution of the formulas used are readily traced without any further description. The weights q are based on an assumed weight of the concrete of 2 300 kls. per cubic meter.

Table No. 2 gives the general data relative to dead loads, and the computation of the line of thrust for a live load of 200 kls. per square meter over the entire span. This case of loading should give a line of thrust corresponding with the center line of the arch ring.

Table No. 3 gives the computation for the values of maximum y, being the result of loading which produces the maximum compression in the intrados, and the minimum stress in the extrados.

Table No. 4 gives the computation for the values of minimum y, being the result of loading which produces the maximum compression in the extrados and the minimum stress in the intrados.

Table No. 5 gives the computation of the thickness of the arch ring at the several points selected, also of the ordinates of the center line, and the unit stresses in the extreme fibers of the arch ring. See Plate XLVI for graphical representation.

TABLE No. 1.—PRELIMINARY INVESTIGATION. LIVE LOAD OF 400 KLS. PER METER, AND CONCENTRATED LOAD OF 10 000 KLS.

Point.	q	r	$q\,r$	Σq	$\Sigma q\,r$	$\frac{\Sigma qr}{\Sigma q} - b$	e	a	β	From Eqs. (32). H	From Eqs. (32). V	From Eqs. (32). $y_{max.}$	S Eq. (33).	N_i Eq. (36).	From Eqs. (34). H	From Eqs. (34). V	From Eqs. (34). $y_{min.}$	S Eq. (35).	N_e Eq. (36).	D Eq. (40).	u Eq. (41).	Y_c Eq. (42).
	kls.	m.	kl. m.	kls.	kl. m.	m.	m.	m.		kls.	kls.	m.	kls.	kls.	kls.	kls.	m.	kls.	kls.	cm.	cm.	m.
1..	8 000	1.35	10 800																			
2..	9 120	4.2	38 304																			
3..	10 840	7.2	78 048	27 960	127 152	4.55	4.2	8.7	5° 40'	312 198	7 670	0.621	37 313	314 320	302 652	6 600	0.207	23 160	303 424	143.6	21.5	0.423
4..	12 400	10.2	126 480																			
5..	13 700	13.2	180 840																			
6..	14 600	16.2	236 520	68 660	670 992	9.77	8.3	17.7	15° 10'	314 637	6 993	2.266	78 495	324 112	295 713	4 673	1.621	67 747	303 045	166.0	33.0	1.964
7..	15 910	19.2	305 472																			
8..	17 220	22.2	382 284																			
9..	18 120	25.2	456 624																			
10..	19 080	28.2	538 056	138 990	2 353 428	16.93	13.0	29.7	26° 30'	317 020	6 330	6.590	150 526	350 550	287 330	2 344	6.150	143 326	320 796	150.0	22.0	6.394
11..	20 320	31.2	633 984																			
12..	22 840	34.2	781 128	182 150	3 768 540	20.689	....	36.0	33° 30'													

$\Sigma_0^{\frac{l}{2}} q = 182\,150.$ $\qquad S = 15\,311.$

Constants .... $\frac{s}{f}\,\Sigma_0^{\frac{l}{2}} q = 278\,890.$ $\qquad \frac{p\,l^2}{8} = 259\,200.$ $\qquad W\left(\frac{l}{2} - 1\right) = 350\,000.$

Case of maximum loading. Entire span covered with 400 kls. per meter and 10 000 kls. at the crown.

Equation (26) ...... $H_{max} = \frac{s}{f}\,\Sigma_0^{\frac{l}{2}} q + \frac{p\,l^2}{8\,f} + \frac{W\,l}{4\,f} = 322\,810$ kls., and D_o at crown $= \frac{H_{max}}{k} = \frac{3228.1}{40} = 80.7$ cm. $+ 10\% = 88$ cm.

Equation (27) .... $S_{max} = \Sigma_0^{\frac{l}{2}} q + \frac{p\,l}{2} + \frac{W}{2} = 201\,550$ kls.

Equation (36) .... $N_{max} = H_{max} \cos \beta_{12} + S_{max} \sin \beta_{12} = 394\,760$ kls., and D_{12} at springing $= \frac{N_{max}}{k} = \frac{3947.6}{40} = 98.7$ cm., or say 106 cm.

TABLE No. 2.—General Data Relative to Dead Loads and Computation of Line of Thrust for Live Load of 200 Kls. per Square Meter over Bridge.

Point.	q	r	$q\,r$	$\Sigma_o^a\,q$	$\Sigma\,q\,r$	$\frac{\Sigma\,q\,r}{\Sigma\,q} = b$	a	c	β	Sin β	Cos β	$a - b$	$(a-b)\Sigma_o^a\,q$	$\frac{a^2\,p}{2}$	y Eqs. (26).
	kls.	m.	kl. m.	kls.	kl. m.	m.	m.	m				m.	kl. m.	kl. m.	m.
1......	8 800	1.35	11 880	8 800	11 880	1.35	2.7	1.4	1° 00'	0.017	0.9998	1.35	11 880	729	0.042
2......	9 710	4.2	40 782	18 510	52 662	2.86	5.7	2.8	2° 50'	0.050	0.999	2.84	52 568	3 249	0.185
3......	11 440	7.2	82 368	29 950	135 030	4.51	8.7	4.2	5° 10'	0.090	0.996	4.19	125 490	7 569	0.442
4......	12 860	10.2	131 172	42 810	266 202	6.22	11.7	5.6	8° 10'	0.142	0.990	5.48	234 599	13 689	0.824
5......	13 630	13.2	179 916	56 440	446 118	7.90	14.7	6.9	11° 10'	0.194	0.981	6.80	383 792	21 609	1.346
6......	14 940	16.2	242 028	71 380	688 146	9.65	17.7	8.1	14° 00'	0.242	0.970	8.05	574 609	31 329	2.012
7......	15 980	19.2	306 816	87 360	994 962	11.39	20.7	9.3	16° 50'	0.290	0.957	9.31	813 322	42 849	2.842
8......	17 260	22.2	383 172	104 620	1 378 134	13.17	23.7	10.5	20° 10'	0.345	0.939	10.53	1 101 649	56 169	3.844
9......	18 460	25.2	465 192	123 080	1 843 326	14.98	26.7	11.6	23° 20'	0.396	0.918	11.72	1 442 498	71 289	5.026
10	19 700	28.2	555 540	142 780	2 398 866	16.80	29.7	12.7	26° 30'	0.446	0.895	12.90	1 841 862	88 209	6.408
11......	20 960	31.2	653 952	163 740	3 052 818	18.64	32.7	13.8	29° 50'	0.497	0.867	14.06	2 302 184	106 929	7.998
12......	22 520	34.2	770 184	186 260	3 823 002	20.525	36.0	14.8	33° 30'	0.552	0.834	15.475	2 882 374	129 600	10.000

$$s = \frac{l}{2} - b_{12} = 15.475$$

Case of maximum loading, for live load $p = 400$ kls. per square meter over entire span, and $W = 10\,000$ kls. at crown.

Constants $\quad \frac{s}{f}\ \Sigma_o^{\frac{l}{2}}\,q = 288\,237 \qquad p\,\frac{l^2}{8} = 259\,200 \qquad W\left(\frac{l}{2} - 1\right) = 350\,000.$

From equation (26). $\quad H_{max} = \frac{s}{f}\ \Sigma_o^{\frac{l}{2}}\,q + p\,\frac{l}{8} + \frac{W\,l}{4\,f} = 332\,157$ kls., from which $D_o = \frac{3321.57}{40} = 83$ cm.

88 cm. was taken.

From equation (27). $\quad S_{max} = \Sigma_o^{\frac{l}{2}}\,q + p\,\frac{l}{2} + \frac{W}{2} = 205\,660$ kls.

From equation (36). $\quad N_{max} = H_{max}\cos\beta_{12} + S_{max}\sin\beta_{12} = 390\,543$ kls., from which $D_{12} = \frac{3905.4}{40} = 97.6$ cm.

106 cm. was taken.

From equation (37). $\quad T = S\cos\beta - H\sin\beta = -11\,860$ kls.

TABLE No. 3.—Values of y_{max}, N_i and T_i.

Point.	$\frac{p\,e}{2\,l}(l-e)$	$\frac{p\,e}{4f}(l-e)$	$\frac{p}{2\,l}\left(\frac{l}{2}+e\right)^2$	$p\,e\left(a-\frac{e}{2}\right)$	$a\,V$	$(a-b)\,\Sigma_o^a\,q$. from Table No 2.	H	y_{max}.	$\Sigma_o^a\,q$. from Table No. 2.	S	$H \cos\beta$	$S \sin\beta$	N_i	T_i
					kl. m.	kl. m.	kls.	m.	kls.	kls.	kls.	kls.	kls.	kls.
1.....	274.0	986	3 884	2 160	22 105	11 880	319 683	0.113	8 800	17 545	319 621	298	319 919	12 106
2.....	538.3	1 938	4 181	4 816	45 159	52 568	320 635	0.320	18 510	27 552	320 315	1 377	321 692	11 493
3.....	791.0	2 847	4 488	11 088	66 729	125 490	321 544	0.632	29 950	39 299	320 258	3 537	323 785	10 203
4.....	1 032.9	3 718	4 806	19 936	86 909	234 599	322 415	1.059	42 810	52 477	319 191	7 452	326 643	6 169
5.....	1 247.8	4 492	5 111	30 050	106 034	383 792	323 189	1.609	56 440	66 412	317 048	12 884	329 932	2 452
6.....	1 437.8	5 176	5 401	44 226	124 311	574 609	323 873	2.294	71 380	81 642	314 157	19 757	333 914	816
7.....	1 619.8	5 831	5 699	59 706	141 613	813 322	324 528	3.127	87 360	97 920	310 573	28 397	338 970	— 404
8.....	1 793.8	6 458	6 004	73 290	158 013	1 101 649	325 155	4.099	104 620	115 485	305 320	39 842	345 162	— 3 738
9.....	1 946.3	7 006	6 292	96 976	173 942	1 442 498	325 703	5.261	123 080	134 233	298 995	53 156	352 151	5 752
10.....	2 092.0	7 531	6 586	118 618	189 159	1 841 862	326 228	6.590	142 780	154 227	291 974	68 785	360 759	— 7 465
11.....	2 231.0	8 032	6 887	142 416	203 721	2 302 184	326 729	8.106	163 740	175 488	283 274	87 217	370 491	— 10 236
12.....	2 351.6	8 466	7 166	169 312	219 938	2 882 374	327 163	10.000	186 260	198 287	272 854	109 454	382 308	—15 223

Equations Used.

Equations.... (32)

$$V = \frac{1}{l}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2} - 1\right) - \frac{p\,e}{2}(l-e)\right] = 8\,461 - \frac{p\,e}{2\,l}(l-e)$$

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\,q + \frac{1}{2\,f}\left[\frac{p\,l^2}{8} + W\left(\frac{l}{2}-1\right) + \frac{p\,e}{2}(l-e)\right] = 318\,697 + \frac{p\,e}{4\,f}(l-e)$$

$$y_{max} = \frac{(a-b)\,\Sigma_o^a\,q + a\,V + e\,p\left(a - \frac{e}{2}\right)}{H}$$

Equation..... (33) $S = \frac{p}{2\,l}\left(\frac{l}{2} + e\right)^2 + \frac{W}{l}\left(\frac{l}{2} - 1\right) + \Sigma_o^a\,q.$

Equation..... (36) $N_i = H \cos\beta + S \sin\beta.$

Equation..... (37) $T_i = S \cos\beta - H \sin\beta.$

Data.

Values of a, e, $\sin\beta$ and $\cos\beta$ from Table No. 2.

$\frac{l}{2} = 36$ m. and $f = 10.0$ m.

$p = 400$ kl. per square meter and $W = 10\,000$ kls.

$\frac{s}{f}\,\Sigma_o^{\frac{l}{2}}\,q = 288\,237$ and $\frac{W}{l}\left(\frac{l}{2} - 1\right) = 4\,861.$

TABLE No. 4.—Values of y_{min}, N_e and T_e.

Point.	$\frac{p}{2}\left(\frac{l}{2}-e\right)^2$	$W\left(\frac{l}{2}-a\right)$	V	H	$p\,(a-e)$	$\frac{p}{2}\,(a-e)^2$	$(a-b)\,\Sigma_o^a q$ from Table No. 2.	$a\,V$	$y_{min.}$	$\Sigma_o^a q$ from Table No. 2.	S	$H\cos\beta$	$S\sin\beta$	N_e	T_e
			kls.	kls.			kl. m.	kl. m	m.	kls.	kls.	kls.	kls.	kls.	kls.
1	239 432	333 000	7 950	316 859	520	338	11 880	21 465	—0.029	8 800	1 370	316 797	23	316 820	—4 016
2	220 448	303 000	7 270	314 409	1 160	1 682	52 568	41 439	+0.047	18 510	12 400	314 095	620	314 715	—3 332
3	202 248	273 000	6 600	311 999	1 800	3 872	125 490	57 420	0.231	29 950	25 150	310 751	2 263	313 014	—3 030
4	184 832	243 000	5 943	309 629	2 440	7 442	234 599	69 533	0.557	42 810	39 307	306 533	5 582	312 115	—5 053
5	169 362	213 000	5 310	307 355	3 120	12 168	383 792	78 057	1.034	56 440	54 250	301 515	10 524	312 039	—6 408
6	155 682	183 000	4 704	305 171	3 840	18 432	574 609	83 261	1.670	71 380	70 516	296 016	17 065	313 081	—5 450
7	142 578	153 000	4 105	303 016	4 560	25 992	813 322	84 973	2.489	87 360	87 815	289 986	25 466	315 452	—3 835
8	130 050	123 000	3 515	300 889	5 280	34 848	1 107 649	83 305	3.500	104 620	106 385	282 535	36 703	319 238	—3 911
9	119 072	93 000	2 945	298 840	6 040	45 602	1 442 498	78 631	4.713	123 080	126 175	274 335	49 965	324 300	—2 512
10	108 578	63 000	2 383	296 816	6 800	57 800	1 841 862	70 775	6.162	142 780	147 197	265 650	65 650	331 300	—1 219
11	98 568	33 000	1 827	294 815	7 560	71 442	2 302 184	59 743	7.848	163 740	169 473	255 605	84 228	339 833	+ 410
12	89 888	000	1 248	292 731	8 480	89 888	2 882 374	44 928	10.000	186 269	193 492	244 138	106 807	350 945	— 215

Equations used.

Equations 34..

$$V = \frac{1}{l}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right]$$

$$H = \frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q + \frac{1}{2f}\left[\frac{p}{2}\left(\frac{l}{2}-e\right)^2 + W\left(\frac{l}{2}-a\right)\right]$$

$$y_{min} = \frac{(a-b)\ \Sigma_o^a\ q - a\,V + \frac{p}{2}\,(a-e)^2}{H}$$

Equation 35... $S = \Sigma_o^a\, q + p\,(a-e) - V.$

Equation 36... $N_e = H\cos\beta + S\sin\beta.$

Equation 37... $T_e = S\cos\beta - H\sin\beta.$

Data.

Values of a, e, $\sin\beta$ and $\cos\beta$ are taken from Table No. 2.

$\frac{l}{2} = 36$ m. and $f = 10.0$ m.

$p = 400$ kls. per square meter and $W =$ 10 000 kls.

$\frac{s}{f}\,\Sigma_o^{\frac{l}{2}} q = 288\ 237.$

Point.	Δy.	Cos β. Table No. 2.	c.	N_i Table No. 3.	N_e Table No. 4.	$\dfrac{N_e}{k\left(1+\dfrac{N_i}{N_e}\right)}$	$\left[\dfrac{N_i}{k\left(1+\dfrac{N_i}{N_e}\right)}\right]^2$	$6c\left[\dfrac{N_i}{k\left(1+\dfrac{N_i}{N_e}\right)}\right]$	D.	$\dfrac{kD}{N_e}-1$	w.	$y_{min.}$ Table No. 4.	y_c	w^1	Unit Stresses Caused by N_e: $k_e=\dfrac{N_e}{D}\left(1+\dfrac{6w}{D}\right)$	Unit Stresses Caused by N_e: $k_i=\dfrac{N_e}{D}\left(1-\dfrac{6w}{D}\right)$	Unit Stresses Caused by N_i: $k_e=\dfrac{N_i}{D}\left(1-\dfrac{6w'}{D}\right)$	Unit Stresses Caused by N_i: $k_i=\dfrac{N_i}{D}\left(1+\dfrac{6w'}{D}\right)$
	cm.		cm.	kls.	kls				cm.		cm.	m.	m.	cm.	atm.	atm.	atm.	atm.
1..	14.2	0.999	14.2	3 200	3 168	39.8	1 584.0	3 390.9	110.4	0.39	7.2	−0.029	0.043	7.0	+39.9	+17.50	+17.97	+40.0
2..	27.3	0.999	27.3	3 217	3 147	39.8	1 584.0	6 519.2	129.8	0.65	14.1	+0.047	0.188	13.2	40.0	+ 8.48	+ 9.67	40.0
3..	40.1	0.996	40.1	3 238	3 130	39.8	1 584.0	9 575.8	145.4	0.86	20.8	0.231	0.440	19.3	40.0	+ 3.00	+ 2.21	40.0
4..	50.2	0.990	50.1	3 266	3 121	39.9	1 592.0	11 993.9	156.4	1.00	26.1	0.557	0.820	24.0	40.0	− 0.06	+ 1.67	40.0
5..	57.5	0.981	56.4	3 299	3 120	40.1	1 608.1	13 569.8	163.3	1.10	29.9	1.034	1.339	26.5	40.1	− 1.89	+ 0.53	39.9
6..	62.4	0.970	60.5	3 339	3 131	40.4	1 632.2	14 665.2	168.0	1.15	32.2	1.670	2.002	28.3	40.0	− 2.80	− 0.20	40.0
7..	63.8	0.957	61.1	3 390	3 154	40.8	1 664.6	14 957.3	169.7	1.15	32.5	2.489	2.828	28.6	40.1	− 2.80	− 0.20	40.0
8..	59.9	0.939	56.2	3 452	3 192	41.4	1 714.0	13 960.1	166.6	1.09	30.3	3.500	3.833	25.9	40.0	− 1.72	+ 1.44	39.9
9..	54.8	0.918	50.3	3 522	3 243	42.2	1 780.8	12 735.9	162.7	1.01	27.3	4.713	5.010	23.0	40.0	− 0.14	+ 3.24	40.0
10..	42.8	0.895	38.3	3 608	3 313	43.2	1 866.2	9 927.4	151.8	0.83	21.0	6.162	6.397	17.3	39.9	+ 3.61	+ 7.60	40.0
11..	25.8	0.867	22.4	3 705	3 398	44.3	1 962.5	5 953.9	123.3	0.57	12.7	7.848	7.994	9.8	40.0	+10.97	+15.57	40.0
12..	00	0.834	00	3 823	3 509	....	....	....	......	......	0.0	10.000	10.000	0.0	....	....	....	....

EQUATIONS USED.

$$\Delta y = y_{max} - y_{min}$$
$$c = \Delta y \cos \beta.$$

Equation (40). $$D = \sqrt{6c\left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right] + \left[\frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}\right]^2} + \frac{N_i}{k\left(1+\frac{N_i}{N_e}\right)}$$

Equation (41). $$w = \frac{D}{6}\left(\frac{kD}{N_e} - 1\right)$$

Equation (42). $$y_c = y_{min} + \frac{w}{\cos\beta}$$

Equation (39). $$k_1 = \frac{N}{D}\left(1 - \frac{6w}{D}\right) \text{ and } k_2 = \frac{N}{D}\left(1 + \frac{6w}{D}\right)$$

The values of N_i and N_e in the above table are given for an arch 1 cm. wide, thus giving unit stresses in atmospheres of 1 kl. per cm.² for k_e and k_i.

w = distance from the point of application of N_e to the arch center line.

w' = $c - w$ = distance from the point of application N_i to the arch center line.

y_c = ordinate of the arch center line.

k = allowable unit stress on the extreme fiber 40 atm. in compression; 2 atm. in tension (See also Fig. 16).

The pressure on the abutment foundation is given on Plate XLVI.

TABLE No. 6.—CHANGE IN LENGTH OF THE ARCH RING.

Point.	$\Sigma_o^a q$ from Table No. 2.	$a p$	S	$H \cos \beta$	$S \sin \beta$	N	F	L	$\frac{N}{F}$	$\frac{N L}{F}$	$\frac{N L}{F}\left(\frac{1}{E}+\frac{1}{E'}\right)$	$L(\varepsilon + \alpha t)$.	ΔL.
	kls.	kl. m.	kls.	kls.	kls.	kls.	cm².	cm.	atm.		cm.	cm.	cm.
0......	000	00	00	301 197	000	301 197	9 360	135	32.2	4 347	—0.0162	—0.1499	—0.1661
1......	8 800	540	9 340	301 137	159	301 296	11 040	285	27.3	7 780	—0.0290	—0.3163	—0.3453
2......	18 510	1 140	19 650	300 896	983	301 879	12 980	300	23.3	6 990	—0.0260	—0.3330	—0.3590
3......	29 950	1 740	31 690	299 993	2 852	302 845	14 580	301	20.8	6 261	—0.0233	—0.3341	—0.3574
4......	42 810	2 340	45 150	298 185	6 411	304 596	15 640	304	19.5	5 928	—0.0221	—0.3374	—0.3595
5......	56 440	2 940	59 380	295 474	11 520	306 994	16 330	306	18.8	5 753	—0.0214	—0.3397	—0.3611
6......	71 380	3 540	74 920	292 161	18 131	310 292	16 800	309	18.5	5 717	—0.0213	—0.3430	—0.3643
7......	87 360	4 140	91 500	288 246	26 535	314 781	16 970	314	18.5	5 809	—0.0216	—0.3485	—0.3701
8......	104 620	4 740	109 360	282 824	37 729	320 553	16 660	319	19.2	6 125	—0.0228	—0.3541	—0.3769
9......	123 080	5 340	128 420	276 499	50 854	327 353	16 270	325	20.1	6 532	—0.0243	—0.3607	—0.3850
10......	142 780	5 940	148 720	269 571	66 329	335 900	15 180	333	22.1	7 359	—0.0274	—0.3696	—0.3970
11......	163 740	6 540	170 280	261 138	84 629	345 767	13 330	342	26.9	9 200	—0.0342	—0.3796	—0.4138
12......	186 260	7 200	193 460	251 198	106 790	357 988	11 280	216	31.7	6 847	—0.0255	—0.2398	—0.2653
									Average = 23.0				Total δ = —4.5208

EQUATIONS USED.

Equation . . (26). $H = \frac{s}{f} \Sigma_o^{\frac{l}{2}} q + \frac{p l^2}{8 f} = 301\ 197$ kls. Equation (36). $N = H \cos \beta + S \sin \beta$.

Equation . . (27). $S = \Sigma_o^a q + a p$. Equation . . (46). $\triangle L = -\frac{N L}{F}\left(\frac{1}{E} + \frac{1}{E'}\right) - L\left(\varepsilon + \alpha t\right)$

A comparison of the last assumed data, and the final dimensions obtained in Table No. 5 is given in the following table, from which it is clearly seen that the last assumed arch ring gave dead loads and dimensions which could scarcely be improved by further computations:

Point.	Assumed Dimensions.		Computed Dimensions.		For ½ total live load as found in Table No. 2.
	D.	y_c	D.	y_c	y.
	cm.	m.	cm.	m.	m.
0	88	0.00	88	0.000	0.000
1	112	0.05	110.4	0.043	0.042
2	131	0.18	129.8	0.188	0.185
3	145	0.45	145.8	0.440	0.442
4	154	0.80	156.4	0.820	0.824
5	162	1.29	163.3	1.339	1.346
6	166	1.96	168.0	2.002	2.012
7	167	2.80	169.7	2.828	2.842
8	166	3.79	166.6	3.833	3.844
9	160	4.98	162.7	5.010	5.026
10	150	6.38	151.8	6.397	6.408
11	134	7.96	133.3	7.994	7.998
12	106	10.00	106.0	10.000	10.000

The above agreement between y_c and y illustrates very strikingly the statement previously made, viz.: that the line of thrust, for a case of entire span covered with one-half the uniform live load, represents practically the center line of the arch.

It will be noticed that the tensile stresses on the intrados at points 6 and 7 (see Table No. 5) are slightly in excess of the allowable, being 2.8 atm., as against 2 atm. allowed. This would probably be safe, but having set the limit at 2 atm., the excess of 0.8 atm. must be provided for, either by giving the section larger dimensions at these points, or by introducing a few iron rods to take up the excessive tension. The latter method will be adopted merely to illustrate the application of the formula.

The sectional area of iron required for an arch of 1 cm. width is by

$$\text{equation (45), } a = \frac{k_1^2 D \left(\frac{D}{2} - \frac{k_1 D}{3 (k_1 + k_2)}\right)}{2 f v (k_1 + k_2)}$$

in which $k_1 = 0.8$ atm., $k_2 = 40$ atm., $D = 169.7$ cm., $f = 700$ atm. (10 000 lbs. per square inch), and $v = 75$ cm., leaving about 10 cm. of concrete outside the iron.

By substituting these values in (45), and solving, it is found that $a = 0.002$ cm.^{2}, or for an arch 1 m. wide 0.2 cm.^{2} would be required,

or 3 rods of 3 mm. diameter for each meter of arch, or 7 rods of 2 mm. diameter, whichever may be preferable.

(*f*) *Design of the Hinged Bearings.*—The radius of curvature of the rolling surface according to Winkler, Heinzerling or Melan is given by $r = \frac{2\,N}{\pi\,k\,l}$ in which $N =$ the maximum normal pressure on the bearing $=$ 390 543 kls. for 1 m. width of arch, $l =$ length of rolling surface $=$ 70 cm. and $k =$ allowable unit working stress $=$ 240 atm. for cast iron under slow motion.

Then

$$r = \frac{2 \times 390\,543}{3.14 \times 240 \times 70} = 15 \text{ cm.}$$

To find the height of the bearing necessary to distribute the stress over the concrete, let

$h =$ height of the bearing.

$D =$ thickness of arch ring adjacent to the bearing $=$ 106 cm.

$b =$ width of bearing $=$ 100 cm., but the rolling surface is only 70 cm.

$N =$ normal pressure on the bearing for 1 m. width of arch $=$ 390 543 kls.

$T =$ tangential stress on the bearing for 1 m. width of arch $=$ 11 860 kls.

$M =$ maximum bending moment on the bearing.

$I = \frac{b\,h^3}{12} =$ moment of inertia of vertical section through axis of roller.

$k =$ allowable unit stress $=$ 500 atm. for cast iron, quiescent load.

Then

$$M = \frac{N}{D} \times \frac{D^2}{8} = \frac{N\,D}{8}; \text{ also } M = \frac{k\,I}{\frac{h}{2}} = \frac{N\,D}{8}$$

which solved for h gives

$$h = \sqrt{\frac{3\,N\,D}{4\,k\,b}} = \sqrt{\frac{3 \times 390\,543 \times 106}{4 \times 500 \times 70}} = 30 \text{ cm.}$$

The coefficient of friction between cast iron and stone is about 0.6; hence a tangential stress of 0.6 $N =$ 234 300 kls. would be required to

slide the bearing plate. The existing tangential stress is only 11 860 kls. However, a small rib is provided on the bottom of the plate to prevent sliding during the erection of the structure.

To diminish the sliding friction between the roller surfaces of the bearings, it is intended to make the radius of the convex surface 15 cm. and that of the concave surface 16.5 cm. (not shown on the drawings), thus converting the sliding friction into rolling friction. The arc described by these rollers, for the extreme movements of the arch, is so small that the point of contact between the rolling surfaces would not be appreciably displaced, and there is absolutely no danger of unequal distribution of pressure on the casting, even were the motion to reach 1° of arc.

(*g*) *Composition of the Concrete.*—The full section of the arch ring for a distance of 2 m. from the hinges, and all outside the middle third of the arch ring; also the floor arches and the exposed surfaces of abutments and piers for a depth of 20 cm. from the surface, shall be of concrete, composed of 1 part Portland cement to 2 parts sand to 3 parts limestone. The middle third of the arch, the surfacing of the roadway under the asphalt composition and the cores of the small piers shall be of concrete, composed of 1 to 3 to 6. All other concrete shall be mixed 1 to 4 to 8 except the abutment foundations which shall be made as shown on Plate XLVI.

(*h*) *Camber.*—The camber to be allowed in the arch ring will now be found for the condition that the bridge when completed, carrying the dead load and a live load of 200 kls. per square meter, shall be at its true level at 0° Cent.

To realize this, the falsework must be superelevated by an amount equal to this camber plus the settlement which the former may undergo up to the time of closing the arch ring. The settlement in the falsework should be determined by actual tests made prior to construction.

After the arch ring is closed at mean temperature of, say, 24° Cent., and under no stress, it should be above its geometrical shape by the amount of the camber. Hence, the camber will be equal to the deflection at the crown caused by the dead load and the uniform live load of 200 kls. per square meter, and a diminution in temperature of 24° Cent. below the mean. Then, under ordinary temperatures and loads, the arch will usually be above its theoretical position, which is very

desirable, as the horizontal thrust is materially increased by a diminution in the rise of the arch.

The thrusts N resulting from the assumed case of loading are found in Table No. 6, using equations (26), (27) and (36). The shortening in the successive arch sections between the crown and the abutment, for the respective values of N, are also found in Table No. 6 from equation (46). As the assumed case of loading is one of symmetry, only the half arch is treated.

The concrete to be used for the arch ring should possess an ultimate compressive strength of 220 atm. at probably the age when the bridge will be first tested. From Table No. 6 the average unit working stress, for the case of loading just above mentioned, will be seen to be 23 atm. Then the values of e, α, E and E' can be taken from the data given under Section V (d), as follows: $e = 0.0009$, $\alpha = 0.0000088$, $E = 295\,000$ atm., and $E' = 3\,000\,000$ atm. For $t = -24°$ Cent. $e + \alpha\, t = 0.00111$ and $\frac{1}{E} + \frac{1}{E'} = 0.00000372$, which values are used in Table No. 6 to find the $\triangle L$'s.

The abutments themselves will be somewhat displaced as a result of stress and temperature effect, though the shrinkage will probably have taken place prior to closing the arch ring. Hence the equation for the shortening in the abutment may be written $\triangle L = -\frac{N\,L}{F}\left(\frac{1}{E} + \frac{1}{E^1}\right) - \alpha\, t\, L$. Taking $F = 69\,000$ cm.^{2} as an average value, and $L = 1\,000$ cm., $\triangle L$ becomes 0.23 cm. This displacement is considered in the graphical solution on Plate XLVI, but not in the analytical solution here following:

From Table No. 6, the value of $\delta = \Sigma_0^{\frac{l}{2}} \triangle L = 4.521$ cm., of which 3.693 cm. is permanent and 0.828 cm. is elastic.

Also $n = \Sigma_0^{\frac{l}{2}} L = 3\,789$ cm., $f = 1\,000$ cm., and $\frac{l}{2} = 3\,600$ cm.

Hence from equation (49),

$$C_1 = \left(1 - \frac{\delta}{n}\right)\sqrt{f^2 + \frac{l^2}{4}} = 3\,731.85 \text{ cm.}$$

and from equation (50),

$$\triangle f = f - \sqrt{C_1^2 - \frac{l^2}{4}} = 16.79 \text{ cm.}$$

This agrees very closely with the value 16.65 cm., obtained from the graphical solution on Plate XLVI.

When the displacement of the abutments, amounting to 0.23 cm., is included, the total deflection at the crown will be 17.5 cm. The deflections at any other points of the arch ring may be scaled from the diagram, Plate XLVI.

If the falsework were absolutely rigid, the crown of the arch would require a superelevation of 17.5 cm., so that the arch, if closed at 24° Cent., and when carrying its own weight and a live load of 200 kls. per square meter, will have a rise of 10 m. at 0° Cent. The design of the falsework is not made a part of this problem.

(*i*) *The Pressure on the Abutment Foundations.*—This is found to be 4.76 atm. (see Plate XLVI). This, for the character of the substrata, assumed in the problem as hard clay, is in no way excessive. However, when dealing with a specific case, the pressure area may be made any desired quantity and the foundation pressure be diminished to such intensity as may seem safe for the particular case.

The fact that small settlements in the abutments of three-hinged masonry arches are not attended by any serious consequences, especially when sufficient camber was put into the arch during construction, makes it perfectly safe to exceed the pressure limits, on foundations hitherto allowed for arches without hinges, by from 50 to 100 per cent.

(*j*) *Estimate of Quantities.*—The following table contains the quantities of Portland cement concrete of the various mixtures for the different parts of the structure.

Structural parts.	Quantities of Concrete, in Cubic Meters.				
	1:2:3	1:2.5:4	1:3:6	1:4:8	1:4:8:3st.
Arch ring	978	....	478	....	....
Floor piers	165	....	52	....	....
Floor arches and floor	328	....	....	....	....
Two abutments and wing walls	141	....	....	1 661	....
Two abutment foundations	....	60	210	....	1 664
Totals in cubic meters	1 612	60	749	1 661	1 664
Totals in cubic yards	2 108.5	78.5	979.7	2 172.6	2 176.5

Estimated Quantities.

Items.	Quantities.	
	Metric units	U. S. units.
Portland cement concrete, mixed, 1c. : 2 s.: 3 broken stone...	1 612 m³.	2108.5 cu. yds.
" " " " 1 : 2.5 : 4 " " ...	60 "	78.5 "
" " " " 1 : 3 : 6 " " ...	749 "	979.7 "
" " " " 1 : 4 : 8 " " ...	1 661 "	2172.6 "
" " " " 1 : 4 : 8 b. s., 3 stone....	1 664 "	2176.5 "
Louisville cement concrete 1 : 6 : 12 broken stone..	720 "	941.8 "
Earth excavation......	5 400 "	7063.0 "
Asphalt pavement over abutments, 15 cm. concrete found...	176 m².	210.3 sq. yds.
Asphalt composition 6 cm. thick over bridge......	600 "	717.0 "
Concrete footwalks over abutments......	88 "	105.2 "
Concrete balustrade over abutments......	44 m.	144.3 ft.
Iron hand railing over bridge......	150 "	492.0 "
194 m. of steel grooved rail, 80 lbs. per yd. or 39.76 kls. per m.	7 713 kls.	16 970 lbs.
972 steel rods, 2.5 cm. dia. 3.05 m. long, nuts each end at 12.4 kls.	12 052 "	26 514 "
120 m. *I* at 14.9 kls. per m. or 10 lbs. per ft......	1 788 "	3 934 "
Metal in expansion joints of roadway......	5 000 "	11 000 "
Cast-iron hinged bearings......	81 600 "	179 520 "
Falsework......	72 lin. m.	236.2 lin. ft.

Unit prices have not been inserted in the above table because these are too much dependent on local conditions and market values. However, a liberal estimate of cost for the entire structure as designed, including 10% for engineering contingencies, would be about $96 000.

MEMOIRS OF DECEASED MEMBERS.

Memoirs will hereafter be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES CHATHAM DUANE, Hon. M. Am. Soc. C. E.*

Died November 8th, 1897.

James Chatham Duane was born at Schenectady, N. Y., June 30th, 1824. He came of old Revolutionary stock; his great grandfather, Judge James Duane, having been prominent in the affairs of the colonies during the struggle for independence from the mother country.

James Duane and John Jay were delegates from New York to the First Continental Congress, but neither of them happened to be in attendance when the Declaration of Independence was signed. James Duane was, however, a prominent member of the committee that drafted the Constitution of the United States.

On the evacuation of New York City by the British troops he was appointed its first Mayor. Duane Street, then the northern limit of the city, was named after him. He was subsequently appointed to a judgeship by President Washington, who had long been his warm personal friend.

The subject of the present obituary notice, James C. Duane, entered Union College in 1840, and was graduated in 1844 with the degree of A. B. He entered the Military Academy at West Point in 1844, was graduated from that institution July 1st, 1848, third in his class, and was at once assigned as an officer of the Corps of Engineers, United States Army. In 1850 he married Harriet W., eldest daughter of Captain, afterwards General, Henry Brewerton, of the Corps of Engineers. United States Army, who was at that time Superintendent of the Military Academy.

He passed through all the grades in the Corps of Engineers, from the lowest to the highest, having reached the office of Chief of Engineers and the grade of Brigadier-General, United States Army, October 14th, 1886. He was retired at the age of sixty-four by operation of law, June 30th, 1888.

His service of forty years in the army, both in peace and in war, was always meritorious and faithful and often distinguished. He received the brevets of Lieutenant-Colonel and Colonel for

"Meritorious and faithful services in the campaign from the Rapidan to the James River, and particularly for distinguished pro-

* Memoir prepared by the following committee appointed by the Board of Direction: Wm. P. Craighill, A. Fteley and James Duane, Members, Am. Soc. C. E.

fessional services before Petersburg, Va.," and the brevet of Brigadier-General "For gallant and meritorious services during the siege of Petersburg, and the campaign terminating with the surrender of the army under General Robert E. Lee."

A detailed statement of General Duane's services is given in the order from the headquarters of the Corps of Engineers soon after his death, which occurred in New York, November 8th, 1897. In concluding this order the Chief of Engineers remarks as follows:

"General Duane's long period of public service was characterized by a faithful devotion to every duty committed to his charge, coupled with professional attainments of the very highest order. A most distinguished officer of the Corps of Engineers, his conspicuous services will be best appreciated by its members, who feel justly proud of his honorable record."

A distinguished comrade in the Corps of Engineers, who knew of what he wrote, remarks as follows:

"After leaving Fort Pickens in 1861, Duane was in Washington in the fall and winter of 1861, engaged in drilling and organizing the Engineer Battalion and in constructing bridge equipage for the army. In 1862, in command of the Engineer Battalion, he went through the Peninsula campaign. At the siege of Yorktown his work was conspicuous. Untiring in energy, after being out on the works nearly all night, he would drop on the ground, covered by his tent, to catch a little sleep, often too fatigued to change clothing soaked by the rain. Regardless of himself, I often wondered how any constitution could endure what he underwent. When the Army of the Potomac left the Peninsula, he built the bridge across the Chicahominy. It was one of the largest pontoon bridges that had ever been built (about 2 000 ft. long), was ready on time, and carried the army without mishap. In the Antietam campaign he was Chief Engineer of the Army of the Potomac and remained such while McClellan was in command. Here, too, he was indefatigable.

"I did not serve in the same field with him again until I returned to the Army of the Potomac in the spring of 1864, finding him again its Chief Engineer. As in the Peninsula and Antietam campaigns, here, again, he was untiring in his professional work, the lines of works about Petersburg, which shut in Lee's army being, perhaps, its most conspicuous example.

"Under trials that would have broken down one less faithful, he met Meade's nervous impatience with his own enduring sense of duty. He had the greatest modesty and that thorough common sense at all times and under all circumstances, which is one of the most valuable intellectual gifts a man can have. Thoroughly true, honest and unselfish, there are few men whose loss will be more deeply felt by those who knew him intimately."

After the war, before General Duane became Chief of his Corps, he had a long service as member and later as President of the Board of Engineers in New York City. His influence was strongly felt in that important Board, and his ideas and suggestions were extensively utilized in the type structures for the defense of our harbors, such as batteries for the heaviest modern guns and mortars, including the gun-lift and turret.

Besides his purely military work, General Duane performed most valuable service as Engineer of the First, Second and Third Lighthouse Districts, and as member of the Lighthouse Board in Washington. An idea of his varied and extensive operations in this professional line may be had from the following summary furnished by officers of the Lighthouse Department.

"Under the direction of General Duane, as Engineer of the First, Second and Third Lighthouse Districts, the following structures have been designed and built:

"Twenty-six cast-iron lighthouses and towers, some of them on cast-iron foundation piers filled with concrete. This kind of structure was introduced by him.

"Fifteen stone and brick lighthouses and towers, among which Fire Island Tower is the most important.

"Sixteen fog-signal stations.

"Thirty-seven keepers' dwellings.

"The most conspicuous features of General Duane's long service under the Lighthouse Establishment were:

"Investigation of the general subject of sound and fog-signal apparatus. See his report of January 12th, 1872, to the Lighthouse Board, for details.

"The practical adaptation of steam whistles for fog-signal purposes. General Duane's designs for the steam-whistle apparatus were the first to be extensively used in the service. At many stations the original apparatus designed by him is still in use, though improvements in many details have since been made. Steam whistles are now used as fog signals at ninety-four stations in the United States service.

"The use of cast-iron concrete-filled foundations, and of cast-iron towers, for light stations. This form of structure has proved economical in first cost, extremely durable and cheap to maintain in repair, and it is particularly well adapted to lighthouses erected in the water in northern latitudes where the structure is exposed to the action of ice. About forty-six lighthouses of this general design have been erected in various parts of the United States."

In 1870–71, while Engineer of the First Lighthouse District, General Duane made an elaborate series of experiments on the transmission of sound, in its application to fog signaling. The subject was then in its infancy, and the theories advanced by the General to account for certain observed anomalies received marked attention from scientists both at home and abroad. His explanations of some perplexing phenomena, at that time little understood and even not generally known, have since been accepted as scientifically correct.

Professor John Tyndall, in his lectures on sound, says: *

"Passing over the record of many other valuable observations in the report of General Duane, I come to a few very important remarks which have a direct bearing on the present question.

"From an attentive observation," writes the General, "during three years, of the fog signals on this coast, and from the report received

* "Tyndall on Sound," 3d Ed., D. Appleton & Co., 1887; also see report of the Lighthouse Board for 1874.

from the captains and pilots of coasting vessels, I am convinced that in some conditions of the atmosphere the most powerful signals will be at times unreliable."*

"Now it frequently occurs that a signal which under ordinary circumstances would be audible at the distance of fifteen miles cannot be heard from a vessel at the distance of one mile. This is probably due to the reflection mentioned by Humboldt.

"The temperature of the air over the land where the fog signal is located being very different from that over the sea, the sound, in passing from the former to the latter, undergoes reflection at the surface of contact. The correctness of this view is rendered more probable by the fact that when the sound is thus impeded in the direction of the sea it has been observed to be much stronger inland.

"Experience and observation lead to the conclusion that these anomalies in the penetration and direction of sound from fog signals are to be attributed mainly to the want of uniformity in the surrounding atmosphere, and that snow, rain and fog, and the direction of the wind have less influence than has generally been supposed."

"The report of General Duane is marked throughout by fidelity to the facts, rare sagacity and soberness of speculation. The last three of the paragraphs quoted exhibit, in my opinion, the only approach to a true explanation of the phenomena which the Washington report reveals."

While thus proving himself an able theorist, the General has conclusively exhibited his ability as a practical mechanical engineer by designing the type of fog-signaling apparatus employed in making the above experiments, and which is still in general use. When he first took charge of the lighthouse work on the New England coast, he was confronted with a difficult problem.

Many of the light stations built early in the century were falling into decay, and the funds available for their renewal were very inadequate. In this emergency he designed a type of tower composed of cast-iron segments bolted together, which, as a model of stability and economy, has been largely used in the service.

He also invented a mixer for concrete which is still extensively used by contractors and engineers on many important works.

General Duane was the author of a manual for Engineer Troops, and he, with General Abbot and Colonel Merrill, of the Corps of Engineers, prepared an important work in 1870 entitled, "Organization of the Bridge Equipage for the U. S. Army."

The following was written by an officer of the Corps of Engineers who had been closely associated with General Duane:

"During the later years of his life his eyesight failed and he was averse to writing, but his mind was a storehouse of useful knowledge which he was always willing to impart.

"His ability and good judgment were so well known that he was frequently consulted on important matters by members of the Corps of Engineers, and his opinions always carried great weight.

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* Had I been aware of its existence I might have used the language of General Duane to express my views on the point here adverted to. See Chap. VII, pp. 319-320.

"He was the most lovable of men and the stanchest of friends. Honest of purpose, he had the faculty of knowing what was right and no sophistry could swerve him.

"His integrity was beyond question and no one who had business relations with him could fail to admire and respect him."

Another officer writes as follows:

"He was quiet, undemonstrative, able and wise, conscientious and perfectly fearless in the discharge of duty. He would always do what he thought right, regardless of personal consequences."

In 1888 General Duane was appointed a member of the Aqueduct Commission of New York. The work, outside of its engineering features, the magnitude of which is well known, presented at that time especial difficulties requiring the highest order of abilities on the part of the commissioners, and General Duane's ripe experience and reliability in council were fully equal to the situation; the value of his service is expressed in the following extract from the resolution passed by the Aqueduct Commissioners:

"General Duane brought to this Commission a mind and disposition exceptionally fitted for the important duties which he was called upon to perform. His designation as a member of this body, in August, 1888, and his immediate election thereafter as its presiding officer, were accepted by the public as guarantees of the spirit with which the great work proposed should be carried out. He more than justified the wisdom of his selection and the confidence of the people. From the day of his appointment until the day of his death, the work of the Aqueduct Commission had his entire and devoted attention; and to his distinguished professional skill, untiring watchfulness and unfailing tact is due in great measure the success with which the work of this Board has been so successfully prosecuted."

The preceding record proves beyond question the great merit and unusual ability of this distinguished soldier and engineer. His noble, lovely qualities as a man endeared him also in an unusual degree to his family and friends, and to them his loss is irreparable.

General Duane was elected an Honorary Member of the American Society of Civil Engineers on November 20th, 1886.

JOHN HOUSTON, M. Am. Soc. C. E.*

Died August 30th, 1896.

John Houston was born at Edinburgh, Scotland, June 24th, 1828. His father was David Houston, a lawyer of that city. He studied in London, and attracted the attention of some prominent engineers, in a public office, by the accuracy of the work entrusted to him. After

* Memoir prepared by John Bogart, M. Am. Soc. C. E.

completing his studies, and after the death of his mother, to whom he was greatly attached, he determined to go to the United States. He had no acquaintances in that country, and he took no letters of introduction ; but he brought with him his sturdy Scotch character and a determination to become a successful engineer. His long professional career and the direction and execution of many important enterprises prove that he was right in the selection of his life work.

In 1853 he made surveys for a proposed ship-canal around Niagara, and in 1854 was engaged in the construction of the water-works at Bridgeport, Conn., and also upon the Brooklyn Water-Works with the late John P. Kirkwood, Past-President, Am. Soc. C. E. During 1855 and 1858, he was in the West, where he located and built the Iowa portion of the Chicago and North Western Railroad and much other railroad work. In 1858 he returned to the Brooklyn Water-Works, and was also engaged upon the Staten Island Railroad. In 1859 he entered the service of the Erie Railway, and was the Chief Engineer of that company for many years. He had entire charge of the construction of the Bergen tunnel, the first of the great double-track tunnels through the high rock formation in New Jersey immediately west of the Hudson River. This was a work of great magnitude and was conducted with energy and skill. It brought the Erie lines to tide-water directly opposite the City of New York. In 1868, under the direction of Mr. Houston, the tunnel was arched with brick. This was done without interrupting the free passage of trains during the prosecution of the work. It is described by Mr. Houston, in one of the early papers of the Society, No. VIII, read by him October 7th, 1868, and published in the first volume of *Transactions*.

Mr. Houston, as the Chief Engineer of the Erie Company, had charge of the extensive improvements made at Long Dock on the New Jersey side of the Hudson River, including also the Pavonia and the New York City ferry slips and bridges. He built the Newburgh branch of the Erie Railway ; located the Wallkill Valley Railway, and was in charge of all the engineering works connected with the operation, maintenance and extension of this great trunk line. He left the Erie road in 1870, and, in 1871, went to Peru as Chief Engineer of the Cerro de Pasco Railway. He was in South America most of the time until 1875, having charge also of the Mejillones and Caracoba Railway and of other works in Bolivia and Peru.

During this period he crossed the Isthmus of Panama eight times. His health was much impaired and he returned to the United States. In 1880 he went to Venezuela as Chief Engineer of the projected railway from La Guayra to Caracas. The preliminary examinations, the surveys, the location and construction of that remarkable road were all under his immediate personal direction. That direction was, under the circumstances, one of much detail, because all of his assist-

ants were natives of the country and without experience in railroad engineering. The location was bold, and the difficulties to be surmounted were peculiar. He finished the work in 1883, and the road has been constantly and successfully operated since that time. He also had charge of the location and construction of the Puerto Cabello and Valentia Railway in Venezuela, and also of similar work in Ecuador.

He returned to the United States after the completion of these works, but was always in delicate health and not able thereafter to undertake the active direction of large enterprises. He was frequently called upon to act as Consulting Engineer, and was a member of various boards and commissions.

Mr. Houston was actively and effectively engaged in important and successful engineering works from his youth until a time when the exposures incident to the climates where much of that work was done unfitted him for active service. He was always devoted to his profession, was a most earnest and conscientious executive, and his aid and advice were very highly valued by the officials of the various companies with which he was connected. His railway constructions are marked achievements in engineering.

Mr. Houston married and is survived by Isabella Atkinson Dorsey, a great grand-daughter of Colonel Thomas Atkinson, of the Army of the Revolution.

He was elected a Member of the American Society of Civil Engineers, May 6th, 1868, and was strongly attached to the Society.

SILVANUS MILLER, Jr., M. Am. Soc. C. E.*

Died December 17th, 1897.

In the death of Silvanus Miller, Jr., the American Society of Civil Engineers lost an able member, and one who had made an enviable reputation in Central and South America. In Central America, especially, he was probably the best known of all the American engineers who have helped in the building of railways and other public works in that country.

Mr. Miller was born on the 3d day of March, 1851, in New Haven, Conn., his father, Silvanus Miller, of New York City, being a railway contractor at that time. His grandfather, Silvanus Miller, also of New York, was for twenty years a Judge of the Supreme Court in that city.

* Memoir prepared by J. T. Norton, M. Am. Soc. C. E.

In 1868 he received his first appointment through a cousin, Mr. M. O. Davidson, at that time Chief Engineer of the New Haven and Derby Railroad.

From 1871 to 1873 he worked for General Nanne in Costa Rica, in charge of a locating party, making surveys for the Costa Rica Railroad, from Port Limon to the interior.

In 1873 he was employed by Henry M. Keith for the survey of the railroad between Leon and Corinto Bay, Nicaragua.

In the spring of 1877 he was appointed General Manager of the "Loma Larga" mines of Salvador by the President of that country, who was their owner.

In 1882 he was engaged as chief of party in the survey of the Santa Ana and Sonsonate Railroad in Salvador.

In the fall of 1883 he was an applicant for the position of Chief Engineer of the Northern Railroad, in Guatemala, the building of which the government of that country was contemplating. He received the appointment and went to work immediately, directing all the studies, surveys, etc., and having his plans accepted.

In June, 1884, he was married in Guatemala City to a young lady of Hungarian parentage, the Baroness Marta de Forckenbeck, who survives him.

In the spring of 1885 the war broke out between Guatemala and Salvador (in which the President of Guatemala, Don Justo Rufino Barrios, was killed), which suspended the railroad work. Mr. Miller then accepted the position of Chief Engineer of the American Dredging Company, on the Panama canal, where he remained until the fall of 1887. This work employed what were, at that time, the largest and most powerful dredges ever operated.

From Panama he went to work on his own mine, "La Chinamita," in Guatemala, where he remained until 1891. In that year José Maria Reyna Barrios (nephew of Justo Rufina Barrios), the late President of Guatemala, who was assassinated on February 8th, 1898, came into power. He at once sent for Mr. Miller to sign the contract for the construction of the first section of the Northern Railroad of Guatemala. He continued the construction of the railroad until it was completed to the end of the fifth section, a distance of 135 miles, in the summer of 1897. Only those who were connected with the actual building of this road will ever appreciate the difficulties met and conquered on the first and second sections, covering the first 50 miles from the coast, and that it ever was built so far is due to the tenacious courage and ability of Mr. Miller and the American contractors who assisted him. Ruin stared him in the face more than once, and the climatic conditions were enough to dishearten the bravest. The work had reached a better climate, and would have speedily been carried to a successful conclusion, had not political troubles precluded the

financing of the scheme. It is a sad commentary that these, together with a gross abuse, on the part of some of his associates and employees, of the implicit trust he placed in them, should have lost to him the financial gain he had so hardly earned.

The five years' residence on the unhealthy coast, together with incessant work and worry, had so undermined his health that, in the fall of 1897, he was forced to go North, hoping that the change would bring a benefit in health. On December 14th, 1897, he caught a slight cold, to which little attention was paid, but after 24 hours he was declared to be dangerously ill with pneumonia, together with heart trouble, of which he died December 17th, at 12 o'clock, at the St. Cloud Hotel, New York City.

Among the government officials and prominent citizens of Guatemala his professional services, as well as his worth socially, were highly appreciated. His death was as deeply and universally regretted by them as by his friends and employees among the Americans and other foreigners in the country. He has left a reputation in this and other republics of Central America of which his relatives and friends may well be proud.

Mr. Miller was elected a Member of the American Society of Civil Engineers on March 2d, 1887.

GOUVERNEUR MORRIS, M. Am. Soc. C. E.

Died December 30th, 1897.

Gouverneur Morris was born in Pottsville, Pa., November 5th, 1847. He was a great-grandson of Robert Morris, the financier of the American Revolution, and a signer of the Declaration of Independence. He attended the private schools of Pottsville where he early developed a predilection for civil engineering, inherited from his maternal grandfather, Samuel Fisher, who was one of the most prominent pioneer engineers of the anthracite coal basins of Pennsylvania, where his two sons, Allen and Howell Fisher, also became conspicuous in the development of the coal and iron industries.

Gouverneur Morris was graduated in 1867 at the Polytechnic College of Philadelphia. In 1868 he began his career, as an Assistant Engineer with the Philadelphia and Reading Railroad, on the Good Spring Extension. From 1870 to 1874 he was Resident Engineer on the Chesapeake and Ohio Railroad, at Huntington, W. Va., Kanawha Division. During 1875 he opened an engineer's office in Charleston, W. Va. In 1876 he was appointed Mining Engineer to the Cannelton

Coal Company. In 1879 he was employed by Professor Leslie on the Second Geological Survey of Pennsylvania. In 1880 the Northern Pacific Railroad Company called on him to exploit the Yellowstone Valley, and their coal and mineral resources. Upon his return from the Northwest, he was appointed, in 1881, Assistant Superintendent of the Lehigh Coal and Navigation Company, resident at Landsford, Pa., which position he held until 1887, when he resigned and became Superintending Engineer for one of the principal contracting firms on the construction of the New Croton Aqueduct.

From 1890 to 1892, he was Chief Engineer of and built the Johnson City and North Carolina Railroad, in Tennessee. During 1893 and 1894, he was engaged in mining coal and iron ore at Norton and Big Stone Gap, Va., and from 1894 until his death he was in the employ of the Philadelphia and Reading Coal and Iron Company, in Detroit, Mich.

His practical energy kept him so constantly in the field, hard at work, that he had little time for literary work, and, excepting in the way of reports, current and professional correspondence, he never wrote anything to keep his memory alive in print; but his works are enduring records of skilful and conscientious execution. Few engineers of his day could comprehend a wider horizon for industrial development or grasp a professional emergency and meet it more quickly and adroitly than he.

In social life he was a universal favorite. His buoyant disposition, cordial manner, sound sense, ready memory and fund of information, his bright conversation, always sparkling with the freshest news, won him friends and admirers wherever he went.

Kindly and generous to a fault, always ready to bear a hand, to help a friend or assist the needy, the recollections of those who had the good fortune to know him, embalm the many sterling qualities which he possessed.

Mr. Morris leaves a widow, who is a great granddaughter of George Walton, of Georgia (also one of the signers of the Declaration of Independence), and one son.

He was elected a Member of the American Society of Civil Engineers on October 3d, 1888.

EDWIN GREEN NOURSE, M. Am. Soc. C. E.*

Died December 8th, 1897.

Edwin Green Nourse was born at Peoria, Ill., February 13th, 1849. His father was Horatio G. Nourse. In early life he came with the family to Moline, Ill., where he attended the public schools. He was

* Memoir prepared by Charles F. Loweth, M. Am. Soc. C. E..

a student of Griswold College, Davenport, Ia., for about two years, from 1869 to 1871, taking a special course in mathematics and scientific branches.

His first employment was as an assistant under the United States Engineers engaged in the improvement of the channel of the Mississippi on the Rock Island Rapids. In the autumn of 1872 he became associated with the Government engineers engaged in the improvement of the Illinois River on the Copperas Creek dam. Soon after this he was employed by the Chicago, Milwaukee and St. Paul Railroad on the construction of its northwestern extension, and remained on this work several years. Upon its completion he was placed in charge of the line from Marion, Ia., to Council Bluffs. Some time later he was Chief engineer of the Chicago and Evanston Railroad, charged with procuring right of way, and the work of construction through Chicago. Later he was connected with the department of maintenance of way of the Santa Fé Railroad on its entrance into Chicago, and was engaged on the city construction of that line. In 1891–93 he was engineer of construction of the terminal station at the World's Fair, Chicago, and after the completion of this work, during the Fair and afterward, was superintendent. He then entered into a business partnership in Chicago, his firm being engaged in general contracting. Subsequently he went to Winona, Minn., where he had previously spent some time and where he had friends. He went to Moline about October, 1897, intending to plat and improve a tract of city property there. About a month later he became assistant engineer of the Davenport and Rock Island Bridge Railway and Terminal Company, and had been at work for this company about two months when he was killed by being struck on the head by the falling mast of a derrick, at Rock Island, Ill., on the morning of December 8th, 1897.

Mr. Nourse was married at Winona, January 21st, 1897. He leaves a widow and an aged father, but no other near relatives.

Mr. Nourse was a prominent mason. He was a member of Blue Lodge and Chapter at Winona, and of Commandery and Consistory at Chicago. His funeral, held at Moline, Friday, December 10th, 1897, was conducted by the Masons of that city. Six civil engineers, members of this Society, acted as honorary pall bearers. The pall bearers, in fact, being old friends of his boyhood.

Mr. Nourse left an unstained record as an engineer of thorough education and qualifications, a man of the highest personal honor, and a gentleman of most agreeable quality. Mr. Nourse was elected a member of the American Society of Civil Engineers on September 3d, 1884.

FRANCIS ENSOR PRENDERGAST, M. Am. Soc. C. E.*

Died December 7th, 1897.

Francis Ensor Prendergast, was born at Dublin, Ireland, October 28th, 1841. He was the only son of John P. Prendergast, Barrister, of Dublin, author of the "Cromwellian Settlement of Ireland," and other Irish historical notes. He came of a very old family that can be traced back to an ancestor who came over to England with the Normans at the time of the Conquest, but he was too modest ever to refer to this himself, thinking that as an American citizen all pride of ancestry should be buried; the family has, however, in fact, held high social position for centuries. He was graduated in Arts and from the School of Engineering of the University of Dublin in 1863. After graduating from college he traveled quite extensively in Germany, France, Switzerland and Italy, and, owing to his keen observation and judgment, his travels were very profitable to him in his professional career.

In 1864 he commenced his engineering practice, after the English fashion, as an apprentice, being articled to the engineer in charge of the Coalbrookdale Railway and Craven Arms Extension Railway, in Shropshire, England. In 1865 he was Assistant Engineer of the City of Glasgow Union Railway, Scotland, and in 1867 Division Engineer on Construction.

He first came to this country in November, 1868, and brought letters of introduction to some of the best people of Boston and New York. He accepted a position as Locating Engineer of the Burlington and Missouri River Railroad in Iowa. In 1870 he was engaged on the preliminary surveys of the Oregon Central Railroad from Astoria to Portland; in 1871, Engineer in charge of construction of the Oregon and California Railroad, between Harrisburg and Pass Creek, 44 miles; in 1872, Locating Engineer of the Oregon Central Railroad, from Forest Grove to Junction City, 100 miles. In 1873 he was Resident Engineer on the construction of the Chicago and Northern Pacific Air Line Railroad, from Geneva Lake to Jefferson, Wis., 35 miles; in 1875, Chief Assistant Engineer of the Chicago and North Eastern Railroad, from Flint to Lansing, Mich. In 1877 he was engaged on surveys for monumenting the City of Omaha. In 1878–79 he was Locating and Resident Engineer on the construction of the Republican Valley Railroad, in Nebraska.

In 1881–82 he was Assistant Engineer on the New York and New England Railroad; in 1884, Locating and Constructing Engineer of the

* Memoir prepared by David W. Cunningham and H. N. Savage, Members, Am. Soc. C. E.

Mahopac Falls Railroad, N. Y. In 1885–86 he was engaged on the Chicago, Burlington and Northern Railroad, as resident engineer of the Galena River Drawbridge, and from East Dubuque to Glen Haven, Wis., 37 miles. In 1887 he was Locating Engineer in Iowa for the same railroad.

He was Assistant Engineer with the New York and New England Railroad, in 1887 and until August, 1888, when he went to Sault Ste. Marie as Resident Engineer for the St. Mary's Falls Water Power Company, remaining until November, 1889, when work was stopped for lack of funds.

In the winter of 1889 he accepted a situation with the San Diego Land and Town Company as Horticultural Superintendent, in which capacity he developed the pioneer details of soil preparation, selection of trees, contour arrangement, setting out and care for the first citrus groves of any magnitude in San Diego County, Cal. In this field, as in all other work undertaken by him, his natural ability and foresight has been highly exemplified; the pioneer methods introduced by him at that time are the universal practice and the most successful at present.

Failing health necessitated a change in residence, and in 1893 he resigned his position and moved to Redlands, Cal., where he resided until his death, his time being taken up with the care of his extensive citrus groves.

Mr. Prendergast was a very attractive writer, and found time from his exacting duties to be a frequent writer for the press, contributing valuable technical papers to the engineering and literary magazines. His work on the details of Railway Construction, published serially in the *Railroad Gazette*, is well known and highly appreciated. He was also a correspondent of Dublin and Belfast, Ireland, newspapers, and contributed three very excellent articles for *Harper's Magazine*, on "Railroads in Mexico," in July, 1881; "The Canadian Pacific Railroad and the New Northwest," in August, 1882, and "Transcontinental Railways," in November, 1883.

He was very careful and accurate in all his work, and his judgment, in all matters that came under his investigation, was of the best. He never gave an opinion that was not carefully considered, and his statements were generally correct; he was, therefore, an excellent adviser. He was respected and beloved by all who knew him for his genial manner and sterling integrity. He was modest and unassuming, and but for his retiring disposition he might have risen to greater distinction.

Mr. Prendergast married Mary A. Childs, of Henniker, N. H., on August 20th, 1873, and leaves his widow and seven children, five boys and two girls.

He was elected a Member of the American Society of Civil Engineers, March 7th, 1888.

EDWARD CURTIS RICE, M. Am. Soc. C. E.*

Died April 21st, 1898.

Edward Curtis Rice, son of Martin and Betsey (Gibbs) Rice, was born July 9th, 1829, in Framingham, Mass. He was educated in the public schools of his native town, attending the Framingham Academy under Marshall Conant, a man of more than ordinary gifts in mathematics, and the Saxonville Academy under the Rev. Mr. Bagnall. From both of these teachers he received valuable instruction preparatory to the profession of a civil engineer, working through the Calculus at nineteen years of age, under Mr. Bagnall. He afterward attended Thetford Academy and there pursued studies fitting him for his life work.

In 1847 Mr. Rice was Assistant to Marshall Conant, Civil Engineer, on the Boston and Cochituate Water-Works under Mr. Chesbrough, Engineer-in-Chief. After the Boston Water-Works were finished, he continued to work and study with Mr. Conant, working on railroads in New Hampshire and Massachusetts.

In 1851 he removed to Dubuque, Ia., and was associated with Sereno Dwight Eaton, Civil Engineer. Since that time he had been engaged in the location and construction of railroads, among which are the following: As Assistant Engineer: on the Mississippi and Atlanta Railroad, the Hannibal and St. Joseph Railroad, and the Keokuk and Des Moines Railroad. As Chief Engineer on the Ohio and Mississippi Railroad, the Cairo and Vincennes Railroad, the Louisville, Evansville and St. Louis Railroad, the Paduca, St. Louis and Chicago Railroad, and the St. Louis, Vandalia and Terre Haute Railroad.

He had as Assistant of late years his nephew, Edward M. Rice.

Early in his engineering work he prepared and published a work called "Tables for Calculating Excavation and Embankment."

In the late Civil War he served as Engineer on the staff of General A. A. Humphreys, 5th Corps of the Army of the Potomac, and on General Mead's staff until November, 1863. He was recommended to General Humphreys by Charles Sumner.

Mr. Rice had lived in St. Louis for nearly thirty years, highly respected by all as a pure and upright man, standing high in his profession, and as a model husband and father. He died on April 21st, 1898, of Bright's disease, and was buried in Bellefontaine Cemetery at St. Louis, Mo.

Mr. Rice was elected a Member of the American Society of Civil Engineers on April 7th, 1875.

*Memoir prepared by George Rice, Esq.

WILLIAM NOYES TAINTOR, Jun. Am. Soc. C. E.*

DIED APRIL 8TH, 1898.

William Noyes Taintor was born in New York City, May 8th, 1870, where he spent all of his early life. He was the son of Mr. and Mrs. Henry F. Taintor, of New York City. He was educated at the Brooklyn Polytechnic Institute and private schools in New York, and at the Columbia College School of Mines, from which he was graduated in 1894. During the summer of 1895 he was Instructor in surveying, in the Columbia College Summer Schools at Litchfield, Conn.

In January, 1896, Mr. Taintor received the second highest general average for the rank of Leveler, at the Civil Service Examinations at Albany, for service on the State canals, and was appointed to the first section of the Oswego Canal, where he was engaged in preliminary surveys and estimates. In June he was given charge of the party which completed the estimates of quantities and prepared the plans for the improvement of that part of the Oswego Canal. In December, 1897, Mr. Taintor was assigned as Engineer in Charge of the construction of the Canaserega culvert of the Erie Canal, where he remained until the following May. In September, 1897, after having passed the Civil Service examinations for a higher office, he was appointed Assistant Engineer on the canal improvement work and assigned as Engineer in charge of contract No. 21, 8½ miles in length, in the vicinity of Oneida, N. Y. While engaged upon this work he contracted a heavy cold which resulted in pleuro-pneumonia, ending after a brief sickness of two weeks in his untimely death at the age of twenty-seven years and eleven months. He was a member of the Alpha Delta Phi fraternity.

Mr. Taintor was deeply interested in his work on the canals, and enjoyed the universal respect of his associates. He had the confidence of his superior officers and of his mates. His favorite study was hydraulics and sanitary engineering, and he found time to devote to the close study of these subjects, even while burdened with the active duties of his position. Tall and commanding in stature, his associates and friends found his heart and loyalty even larger than his frame would promise.

Mr. Taintor was elected a Junior of the American Society of Civil Engineers on September 3d, 1895.

* Memoir prepared by J. C. Wait, M. Am. Soc. C. E.

HISTORICAL SKETCH

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS,

By Charles Warren Hunt, M. Am. Soc. C. E.

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Printed by order of the Board of Direction of the American Society of Civil Engineers, to be sold only on subscription. The proceeds to be devoted exclusively to the fund for the New Society House.

At the Annual Meeting, January 19th, 1898, the following facts in regard to the subscription to this book were brought out:

Two thousand copies were printed; 300 were bound in full morocco, of which 216 have been sold at $10 per copy, the resulting net profit being $943.06. Seventeen hundred copies, which have been paid for, are still on hand, and the Board of Direction was requested to consider the propriety of offering to the membership these copies bound in a less expensive style and at a reduced price, the net proceeds to be applied to the building fund.

In compliance with this request it has been decided to bind as many copies as are necessary to supply the demand, in a handsome cloth binding and to supply them at $5 per copy.

This action has been taken in the belief that many members will welcome the opportunity of contributing something to the building fund.

There are a few copies still on hand of the first lot bound and these can be obtained by those who so desire at $10 per copy.

Orders should be sent to the Secretary.

The book begins with a brief statement of the first movement to form a National Society of American Engineers in 1839. The organization of the American Society of Civil Engineers and Architects in 1852 is then described, a list of its promoters and charter members given, and the work accomplished in its first two years of life sketched. The reorganization of the Association in 1867 and the important events in its career from that date to 1873, when the first publication was issued, are then given in chronological order. Succeeding chapters are under the following heads: Locations Occupied by the Society, Library, International Exhibitions, Publications, Badge, Constitutional Changes and Work Accomplished. Under the head of "Comparative Growth of National Engineering Societies" short sketches of the Institution of Civil Engineers and the Société des Ingénieurs Civils are given. The illustrations consist of 35 half-tone portraits of past officers of the Society and one diagram, all handsomely printed on heavy paper.

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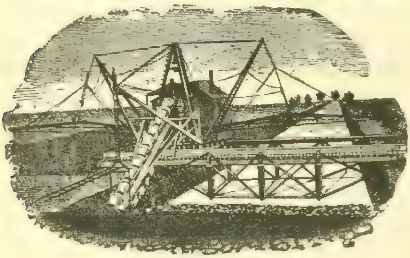

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